CLASSIFICATION ON JOINTED ROCK MASS

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

Master of Technology

In

Civil Engineering

BY

Samaptika Mohanty

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DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY

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Dedicated

To

My Parents

Birendra Kumar Mohanty

&

Kabitanjali Mohanty

NATIONAL INSTITUTE OF TECHNOLOGY
ROURKELA-769008, ODISHA, INDIA

CERTIFICATE

This is to certify that the thesis entitled, **"Classification on Jointed Rock Mass"** submitted by **Ms. Samaptika Mohanty** in partial fulfilment of the requirements for the award of Master of Technology Degree in **CIVIL ENGINEERING** with specialization in **"GEOTECHNICAL ENGINEERING"** at the National Institute of Technology, Rourkela is an authentic work carried out by her under my supervision and guidance.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University / Institute for the award of any Degree or Diploma.

Date:

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Abstract

Rock mass classification is widely used throughout the underground mining industry in both coal and hard rock mines. It is used in all stages of the mining process, from site characterization to production operations. Rock mass characterization is an integral part of rock engineering practice. There is several classification systems used for design of structures on/in rock strata. It is interesting to note that these classification systems: RMi, RMR, Q and GSI, have their origin in civil engineering. Rock mass classification systems are used for various engineering design and stability analysis. These are based on empirical relations between rock mass parameters and engineering applications, such as tunnels, slopes, foundations, and excavatability. Rock mass classification systems have gained wide attention and are frequently used in rock engineering and design. However, all of these systems have limitations, but applied appropriately and with care as they are valuable tools. Different joint configurations will be introduced to achieve the most common modes of failure occurring in nature. A coefficient called Joint factor has been used to account for the weakness brought into the intact rock by jointing. Models have been being prepared using plaster of Paris and Lime-plaster of Paris mix specimens and different degrees of anisotropy have been induced by making joints in them varying from 0 to 90 degree. The specimen will be tested under direct shear, uniaxial compression to determine the various parameters. Rock mass classification system uses rock mass modulus for characterization of systems: RMR, Q, GSI and others. The rock mass classification includes some inputs obtained from intact rock and discontinuity properties, which have major influence on assessment of engineering behaviour of rock mass.

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Nomenclature

J_f	Joint factor
J_n	Number of joints per metre length.
n	Joint inclination parameter
r	Roughness parameter.
β	Orientation of joint.
σ_{cj}	Uniaxial compressive strength of jointed rock.
σ_{ci}	Uniaxial compressive strength of intact rock.
σ_{cr}	Uniaxial compressive ration.
E_{tj}	Tangent modulus of jointed rock
E_i	Tangent modulus of intact rock
E_r	Elastic modulus ratio.
τ	Shear strength
υ	angle of friction
c	Cohesion
Ø	Friction angle
POP	Plaster of Paris
UCS	Uniaxial compressive strength
V_b	Block volume
J_c	Joint condition
J_r	Joint roughness number
J_a	Joint alteration number
J_w	Water reduction factor

CHAPTER - 1

INTRODUCTION

The rocks are formed through very complex processes and the complex behaviour of rock masses is dominated by the planes of weaknesses. Rock mass consists of intact rock and discontinuities.

During the feasibility and preliminary design stage of a project, when very little detailed information is available on the rock mass and its stress and hydrologic characteristics, the use of a rock mass classification scheme can be of considerable benefit. At its simplest, this may involve using the classification scheme as a check list to ensure that all relevant information has been considered, one or, more rock mass classification scheme can be used to build up a picture of the composition and characteristics of a rock mass estimates of support requirements and to provide estimates of the strength and deformation properties of the rock mass. The quality of a rock mass quality can be quantified by means of rock mass classification. Rock mass rating (RMR) system was developed by Bieniaswki 1973. Significant changes have been made over the year with revisions in 1974, 1976, 1979 and 1989. The RMR classification has found wide applications in various types of engineering projects. Such as tunnels, foundations and mines but, not in slopes.

Rocks are not as closely homogeneous and isotropic as many other engineering materials. Rock is confronted as an assemblage of blocks of rock material separated by various types of discontinuities, such as faults, folds, fissures, fractures, joints, bedding planes, shear zones and other structural features which may exert significant influence on their engineering responses. This assemblage constitute a rock mass. The strength of rock masses depends on the behaviour of these discontinuities or planes of weaknesses. The frequency of joints, their orientation with respect to the engineering structures, and the roughness of the joint have a significant importance from the stability point of view. Consequently, the engineering properties of both intact rock and the rock mass must be considered. The properties of the intact rock between the discontinuities and the properties of the joints themselves can be determined in the laboratory where as the direct physical measurements of the properties of the rock mass are very expensive. Artificial joints have been studied mainly as they have the advantage of being reproducible. The anisotropic strength behaviour of shale, slates, and Phillies has been investigated by a large number of investigators. Laboratory studies show that many different failure modes are possible in jointed rock and that the internal distribution of stresses within a jointed rock mass can be highly complex.

The strength of the rock material is included as a classification parameter in the majority of rock mass classification systems. It is a necessary parameter because the strength of the rock material constituents the strength limit of the rock mass. The uniaxial compressive strength of the rock material can be determined in the lab.

Other classification parameters used in current rock mass classifications are spacing of discontinuities, condition of discontinuities i.e. roughness, continuity, separation, joint-wall weathering, infilling. Orientation of discontinuities, groundwater conditions i.e. inflow, pressure, and in-situ stresses.

It is accepted that in the case of surface excavations and those that are controlled by the structural geological features, the following classification parameters are important, strength of intact rock material, spacing of discontinuities, condition of discontinuities, orientation of discontinuities, and groundwater conditions. In the case of deep underground excavations where the behaviour of rock masses is stress can be of greater significance than the geological parameters. Most civil engineering projects, such as tunnels and subway chambers, fall into the first category of geologically controlled rock mass structures.

CHAPTER - 2

LITERATURE REVIEW

2.1 General

Rock mass classifications form the backbone of the empirical design approach and are widely employed in rock engineering. At first, rock classification system given by Terzaghi on the basis of load 40 years ago. Since then, this classification has been modified by Deere et al., 1970 and new rock classification systems have been proposed. Rock mass classifications have been successfully applied throughout the world.

2.2 Jointed Rock Mass

Karl Terzaghi (1946)

Karl Terzaghi analysed that joints and related features are certain types of discontinuities in the rocks. Discontinuity is the general term used in rock mass. It is the collective term for most types of joints, weak bedding planes, weak schistocity planes and faults or weakness zone.

Brekke and Howard (1972)

They suggest the making use of Scale based on aperture, persistence and occurrence and Character based on occurrence of filling material.

Based on this it has in this work been chosen to divide discontinuities in to 3 main groups.

Micro fissures	length < 10 mm
Joints	length 0.1 -100m
Weakness zones	length >30 m

A rock material parameter rather than genuine discontinuity, it is generally included in the properties of rock, therefore it is not further dealt with here.

Arora (1987)

He conducted tests on intact and jointed specimens of plaster of Paris, Jamarani sandstone, and Agra sandstone. Extensive laboratory testing of intact and jointed specimens in uniaxial and triaxial compression revealed that the important factors which influence the strength and modulus values of the jointed rock are joint frequency, joint orientation with respect to major principal stress direction, and joint strength. Based on the results he defined a joint factor (J_f) as,

$$J_f = J_n / (n * r)$$

Where, J_n = number of joints per meter depth

n = inclination parameter depending on the orientation of the joint

r = roughness parameter depending on the joint condition.

Pande, Beer & Williams (1992)

They are analysing the safe design of rock structure such as tunnels; shaft and other underground openings, a symmetric procedure for computing the elasticity matrix of jointed rock mass are used in analysis. The elastic property of the intact rock, rock joints and their orientation and spacing has been available for many years. A reliable determination of elastic normal and tangential stiffness (KN & Ks) for the rock joint has been problematic.

Ming Cai and Hideyuki Horii (1992)

They characterized Rock mass by the existence of distributed joints. The mechanical behaviour of jointed rock masses is strongly affected by the properties and geometry of the joints. The mechanical behaviour of joints is represented by an elasto-plastic constitutive law that is based on the classical theory of plasticity. The effect of the joint connectivity, which results in the reduction of the system stiffness, is treated in the model with a connection co-efficient. The experimental data showing the characteristic features of the behaviour of jointed rock masses.

Grimstad and Barton (1993)

They have also presented an equation to use the Q value to estimate the rock mass deformation modulus (for values of Q > 1). The Q value is also used as one way to estimate the *m* and *s* factors in the Hoek Brown failure criterion (Hoek, 1983; Hoek and Brown, 1988). In this respect, it is only an empirical relationship and has nothing to do with engineering classification.

D.Milne, J.Hadjigeorgiou, R.Pakalnis (1999)

Rock mass characterization is an integral part of rock engineering practice. There are several classification systems used in underground mine design. Three classification systems they are RQD, RMR, and Q system are the origin of the civil engineering. These classification systems as employed in the mining industry. The difference between classification parameters that influence rock mass strength estimation and that influence engineering design is emphasized. Maximum span, opening geometry and support recommendations are based on these systems.

Singh and Goel (1999)

They gave the following comments to the rock load factor classification.

- ⇨ It provides reasonable support pressure estimates for small tunnels with diameter up to 6 meters.
- ⇨ It gives over estimates for large tunnels with diameter above 6 meters.
- ⇨ To estimated support pressure has a wide range squeezing and swelling rock conditions for a meaningful application.

Rao and Tiwari (2002)

In present study an experimental study was planned on specimens of rock mass with three joints sets with varying joint geometry under uniaxial, triaxial and true – triaxial stress state. The physical model testing on jointed rock mass was conducted under uniaxial, triaxial and true – triaxial stress state. The model material was chosen as plaster of Paris and lime with average uniaxial compressive strength 13 MPa.

Sridevi Jade and T.G.Sitharam (2003)

The uniaxial compressive strength of a rock mass has been represented in a non-dimensional form as the ratio of the compressive strength of the jointed rock to the intact rock. The effect of the joints in the rock mass is taken into account by a joint factor. The joint factor is defined as a function of joint frequency, joint orientation, and joint strength. The effect of confining pressure on the elastic modulus of the jointed rock mass is also considered. The models with equivalent properties of the jointed rock mass predict fairly well the behaviour of jointed rock mass.

Hakan Stille and Arild Palmström (2003)

They discussed the role of classification in rock engineering and design. It is important to distinguish between characterization, classification and empirical design method. The main requirements are:

- The reliability of the classes to assess the given rock engineering problem must be estimated.
- The classes must be exhaustive (every object belongs to a class) and mutually exclusive.
- The uncertainties, or the quality, of the indicators must be established so that the probability of miss-classification can be estimated. The useful system should be practical and robust, and give an economic and safe design.

The main classification system is use to fulfil the above requirements. They supervised that the systems as a basis in the development of local system adapted to the actual site conditions.

Ya-ching Liu and Chao-Shi Chen (2006)

They represented a new rock mass classification system. This can be appropriate for rock slope stability assessment. They also evaluated on the combining the analytic hierarchy process (AHP) and the fuzzy Delphi method (FDM) was presented for assessing slope rock mass quality estimates. The linear discriminant analysis (LDA) model was used to classify those that are stable or not; the result show that proposed method can be used to assess the stability of rock slopes according to the rock mass classification procedure and the failure probability in the early stage.

Mahendra Singh, Bhawani Singh, Jaysingh Chaudhary (2006)

The squeezing of tunnels is a common phenomenon in poor rock masses under high in situ stress conditions. The value of critical strain is generally taken as 1%.it is shown in this study that the critical strain is an anisotropic property and that it depends on the properties of the intact rock and the points in the rock mass. A rational classification based on squeezing index (SI) is proposed to identify and quantify the squeezing potential tunnels.

S.Tzamos and A.I Sofianos (2006)

They proposed rock mass classification system by their investigation in this work. They classify rock mass as four types of systems. RMR, Q, GSI, RMI. The common parameters of these systems, which concern and characterize solely the rock mass; are those used for rating the rock structure is quantified by the block size or the discontinuity spacing rating(BS) and the joint surface conditions ratings(JC). A rock mass fabric index denoted as F. Thus it is defined as below:

$$F = F\ (BS, FS)$$

Palmström and Broch (2006)

Their study provides a tool to understand the limitations of rock mass classification scheme and that there use does not replace some of the more elaborate design procedures. These design procedures requires access to relatively detailed information on in situ stresses. Rock mass properties and planned excavation sequence none of which may available at an early stage.

CHAPTER - 3

ROCK MASS CLASSIFICATION

JOINTED ROCK MASS

3.1 General

Jointed rock masses comprise interlocking angular particles or blocks of hard brittle material separated by discontinuity surfaces which may or may not be coated with weaker materials. In geology the term joint refers to a fracture in rock where the displacement associated with the opening of the fracture is greater than the displacement due to lateral movement in the plane of the fracture of one side relative to the other. This makes joints different from a fault which is defined as a fracture in rock in which one side slides laterally past the other with a displacement that is greater than the separation between the blocks on either side of the fracture.

3.2 Joints

Joints normally have a regular spacing related to either the mechanical properties of the individual rock or the thickness of the layer involved. Joints generally occur as sets, with each set consisting of joints sub-parallel to each other. The tensile and compressive stresses which act within the rock are produced due to decrease in volume i.e., shrinkage of the rock mass. These decreases in volume are caused due to:

- Loss of moisture
- Drop in temperature as well as loss of moisture
- Drop in temperature

Due to tensile and compressive stresses in the rock mass regular and irregular cracks or, discontinuities are developed in the rock mass.

Any break in a rock mass irrespective of its size is turned as fracture.

- ➢ Cracks along which the fractured rock mass appear to have suffered no relative displacement are knows as joints.
- ➢ Minor fractures are designated as cracks and fissures
- ➢ Joints occurs in all types of rocks, i.e., igneous, sedimentary and metamorphic.
- ➢ In sedimentary rocks generally there are two systems of mutually perpendicular joints both perpendiculars to bedding planes.

The difficulties of making predictions of the engineering responses of rocks and rock masses derive largely from their discontinuous and variable nature. The strength behaviour of rock mass is governed by both intact rock properties and properties of discontinuities.

The strength of rock mass depends on some factors as follows:

- ♦ The angle made by the joint with the principal stress direction.
- ♦ Opening of the joint
- ♦ The degree of joint separation.
- ♦ Strength along the joint
- ♦ Number of joints in a given direction
- ♦ Joint roughness
- ♦ Joint frequency

This study aims to link between the ratios of intact and joint rock mass strength with joint factor **(J_f)** and other factors. The main important factors which influence the strength and modulus values of jointed rock are

- Joint frequency,**(J_n)**,
- Joint orientation, **β**, with respect to major principal stress direction and joint strength.

These effects can be incorporated into a Joint factor (Ramamurthy (1994)), given as:

$$J_f = J_n / (n * r)$$

3.3 METHODS OF ROCK MASS CLASSIFICATION SYSTEMS

Rock mass systems are classified into four types. They are as follows:

- RMR system
- Q system
- RMi system
- GSI system

3.3.1 RMR SYSTEM

The rock mass rating (RMR) system was initially developed at the South African Council of Scientific and Industrial Research (CSIR) by Bieniawski (1973, 1976, and 1989) on the basis of his experiences in shallow tunnels in sedimentary rocks. This rating system was for use in design of tunnels in hard and soft rock. A revision was made in 1989 to reflect more data collected. To apply the rock mass rating classification system, a given site should be divided into a number of geological structural units in such a way that each type of rock mass is represented by a separate geological structural unit. Six parameters are used to classify a rock mass using the RMR system, that is,

- Uniaxial compressive strength of rock material (A1)
- Rock Quality Designation (RQD) (A2)
- Joint spacing (A3)
- Joint condition (A4)
- Groundwater condition (A5)
- Joint orientation (A6).

3.3.1.1 Uniaxial compressive strength of rock material

The strength of the intact rock material should be obtained from rock cores in accordance with site conditions. UCS may also be obtained from the point load strength index tests on rock lumps at the natural moisture content. The pH value of groundwater may affect the UCS in saturated conditions.

3.3.1.2 Rock Quality Designation

RQD should be determined from rock cores or volumetric joint count. It is the percentage of rock cores acquired equal to or more than 10 cm long in one meter of drill run. The fresh broken cores are fitted together and counted as one piece.

3.3.1.3 Joint spacing

The term “discontinuity” covers joints, beddings or foliations, shear zones, minor faults, and other surfaces of weakness. The linear distance between two adjacent discontinuities should be measured for all sets of discontinuities. It is widely accepted that spacing of joints is very important when appraising a rock mass structure. The very presence of joints reduces the strength of a rock mass and their spacing governs the degree of such a reduction.

3.3.1.4 Joint condition

This parameter includes roughness of discontinuity surfaces, their separation, length of continuity, weathering of the wall rock or the planes of weakness, and infilling material. The joint set, which is oriented unfavourably with respect to a structure, should be considered along with spacing of the discontinuities.

3.3.1.5 Groundwater condition

For tunnels, the rate of inflow of groundwater in litres per minute per 10 m length of the tunnel should be determined, or a general condition may be described as completely dry, damp, wet, dripping, or flowing. If actual water pressure data are available, these should be stated and expressed in terms of the ratio of the seepage water pressure to the major principal stress.

3.3.1.6 Joint orientation

The strike should be recorded with reference to magnetic north. Orientation of discontinuities refers to the strike and dip of discontinuities. The dip angle is the angle between the horizontal and discontinuity plane taken in a direction in which the plane dips. The influence of the strike and dip of discontinuities is considered with respect to the direction of tunnel drive, slope face orientation, or foundation alignment.

The final rating is the summation of all ratings for the six parameters, that is,

$$RMR = A1+A2+A3+A4+A5+A6$$

The RMR value ranges from 0 to 100.

3.3.2 Q SYSTEM

The Q-system, Barton, Lien, and Lunde (1974) at the Norwegian Geotechnical Institute (NGI) originally proposed the Q-system of rock mass classification on the basis of approximately 200 case histories of tunnels and caverns. They defined the rock mass quality (Q) by the following causative factors:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

Where, RQD = Rock Quality Designation,

J_n = Joint set number,

J_r = Joint roughness number,

J_a = Joint alteration number,

J_w = Water reduction factor,

SRF = Stress reduction factor,

For various rock conditions, the ratings of these six parameters are assigned. The goal of the Q-system is to characterize the rock mass and preliminary empirical design of the support system for tunnels and caverns.

In explaining the system and the use of the parameters to determine the value of Q, Barton et al. have given the following explanation:

- The first quotient (RQD/Jn) represents roughly the block size of the rock mass.
- The second quotient (Jr/Ja) describes the frictional characteristics of the rock mass.
- The third quotient (J_w/SRF) represents the active stress situation. This third quotient is the most complicated empirical factor and has been debated in several papers and workshops. It should be given special attention, as it represents four groups of rock masses: stress influence in brittle blocky and massive ground, stress influence in deformable (ductile) rock masses, weakness zones, and swelling rock.

3.3.2.1 Joint set number (J_n)

The parameter J_n, representing the number of joint sets, is often affected by foliations, schistocity, slaty cleavages or beddings, and so forth. Various joint set number (J_n) is specified for different conditions of joints.

3.3.2.2 Joint roughness number and Joint alteration number (J_r and J_a)

The parameters J_r and J_a respectively, represent roughness and degree of alteration of joint walls or filling materials. The parameters J_r and J_a should be obtained for the weakest critical joint set or clay-filled discontinuity in a given zone.

3.3.2.3 Water reduction factor (J_w)

The parameter J_w is a measure of water pressure, which has an adverse effect on the shear strength of joints. This is due to reduction in the effective normal stress across joints. The value of J_w should correspond to the future ground water condition where seepage erosion or leaching of chemicals can alter permeability of rock mass significantly.

3.3.2.4 Stress reduction factor

The stress reduction factor (SRF) parameter is a measure of loosening pressure during an excavation through shear zones and clay-bearing rock masses, rock stress qc/s1 in a competent rock mass where qc is the uniaxial compressive strength (UCS) of rock material and s1 is the major principal stress before excavation, and squeezing or swelling pressures in incompetent rock masses.

The rating for each parameter (except for RQD) is also presented in tables (Barton et al., 1974). For mining application, dry conditions are often assumed and the stress is considered by separate stress modelling so that the modified rock quality index for mining is defined as:

$$Q' = \frac{RQD}{Jn}\frac{Jr}{Ja}$$

The rock mass quality (Q) is a very sensitive index and its value varies from 0.001 to 1000. Use of the Q-system is specifically recommended for tunnels and caverns with an arched roof.

3.3.2.5 Joint orientation and the Q-system

Barton et al. (1974) stated that joint orientation was not as important a parameter as expected, because the orientation of many types of excavation can be, and normally are, adjusted to avoid the maximum effect of unfavourably oriented major joints. The parameters Jn, Jr, and Ja appear to play a more important role than the joint orientation, because the number of joint sets determines the degree of freedom for block movement (if any); the frictional and dilatational characteristics (J_r) can counterbalance the down-dip gravitational component of weight of wedge formed by the unfavourably oriented joints. If joint orientation had been included the classification system would be less general, and its essential simplicity lost.

3.3.2.6 Updating the Q-system

The updating of the Q-system has shown that in the most extreme case of high stress and hard massive rocks, the maximum SRF value has to be increased from 20 to 400 to give a Q-value that correlates with the modern rock supports. With moderately jointed rocks, the SRF needs to be significantly reduced according to the observed tunnelling conditions.

3.3.3 RMi SYSTEM

Palmström (1995) proposed a Rock Mass index (RMi) to characterize rock mass strength as a construction material. The presence of various defects (discontinuities) in a rock mass that tend to reduce its inherent strength are taken care of in Rock Mass index (RMi), which is expressed as

$$RMi = q_{c.} J_P$$

Where, q_c = The uniaxial compressive strength (UCS) of the intact rock material in MPa.

J_P = The jointing parameter composed of mainly four jointing characteristics, namely, block volume or density of joints, joint roughness, joint alteration, and joint size. It is a reduction coefficient representing the effect of the joints in a rock mass. The value of J_P varies from almost 0 for crushed rock masses to 1 for intact rocks = s^n Hoek and Brown's criterion.

RMi = rock mass index denoting UCS of the rock mass in MPa.

The Rock Mass index (RMi) was developed to characterize the strength of the rock mass for construction purpose (Palmström, 1996a, b). RMi is based on the reduced rock strength caused by jointing and is expressed as

$$RMi = 0.2\sigma_c \sqrt{jC}.V_b^{0.37jc^{-0.2}}$$

Where σ_c is the uniaxial compressive strength of intact rock measured on 50 mm samples and V_b is the block volume given in cubic meters and jC is the joint condition, factor expressed as

$$J_c = jL \frac{j_R}{j_A}$$

Where j_L, j_R and j_A are factors for joint length and continuity, joint wall roughness, and joint surface alteration, respectively. Values of RMi range from 0 to σ_c .

3.3.3.1 Selection of parameters used in RMi

If joints are widely spaced or if an intact rock is weak, the properties of the intact rock may strongly influence the gross behaviour of the rock mass. The rock material is also important if the joints are discontinuous. In addition, the rock description includes the geology and the type of material at the site, although rock properties in many cases are downgraded by joints. These considerations and the study of more than 15 different classification systems have been used by Palmström (1995) when selecting the following input parameters for RMi:

1. Size of the blocks delineated by joints measured as block volume, V_b

2. Strength of the block material measured as UCS, q_c

3. Shear strength of the block faces characterized by factors for the joint characteristics, jR and jA

4. Size and termination of the joints given as their length and continuity factor, jL

Table 3.1 Classification of RMi

TERM		
For RMi	**Related to rock mass strength**	**RMi value**
Extremely low	Extremely weak	<0.001
Very low	Very weak	0.001–0.01
Low	Weak	0.01–0.1
Moderate	Medium	0.1–1.0
High	Strong	1.0–10.0
Very high	Very strong	10–100
Extremely high	Extremely strong	>100

3.3.3.2 Applications of RMi

The below figure shows the main areas of RMi application together with the influence of its parameters in different fields. RMi values cannot be used directly in classification systems as many of them are composed of their own systems. Some of the input parameters in RMi are similar to those used in the other classifications and may then be applied more or less directly.

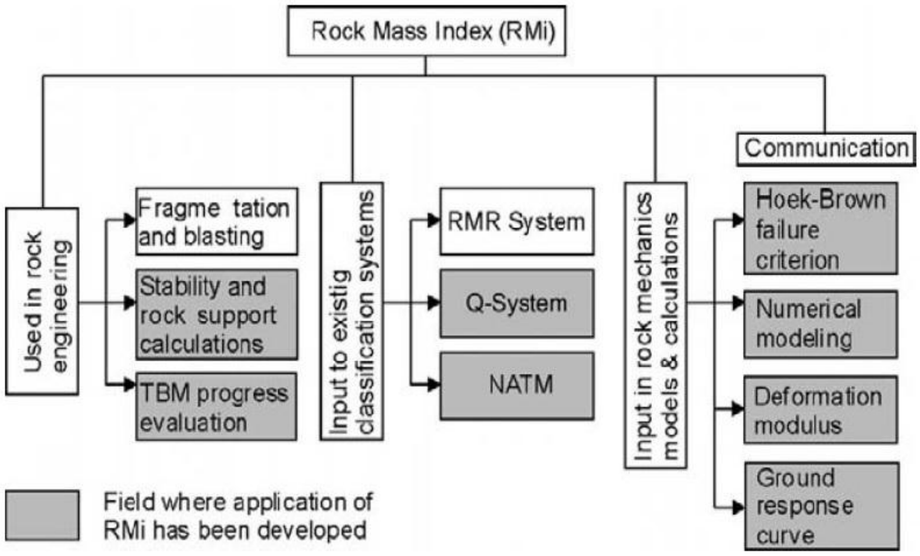


Fig 3.1 Applications of RMi in rock mechanics and rock engineering (From Palmström, 1996).

3.3.3.3 Limitations of RMi

As RMi is restricted to express only the compressive strength of rock masses, it is possible to arrive at a simple expression, contrary to the general failure criterion for jointed rock masses developed by Hoek and Brown (1980) and Hoek et al. (1992).

- ⇨ Both the intact rock material and the joints exhibit great directional variations in composition and structure, which results in an enormous range in compositions and properties for a rock mass. It should be added that RMi probably characterizes a wider range of materials than most other classification systems.
- ⇨ The value of the jointing parameter (JP) is calibrated from a few large-scale compression tests. Both the evaluation of the various factors (jR, jA, and Vb) in JP and the size of the samples tested which in some cases did not contain enough blocks to be representative for a continuous rock mass have resulted in certain errors that are connected to the expression developed for the JP.
- ⇨ The input parameters to RMi express a range of variation related to changes in the actual representative volume of a rock mass. Combination of these variables in RMi may cause errors.

3.3.4 GSI SYSTEM

To provide a practical means to estimate the strength and deformation modulus of jointed rock masses for use with the Hoek-Brown failure criterion (Hoek and Brown 1980, 1988, 1997; Hoek et al. 2002), the GSI was introduced (Hoek et al., 1995). The value of GSI ranges from 0 to 100. The GSI system consolidates various versions of the Hoek–Brown criterion into a single simplified and generalized criterion that covers all of the rock types normally encountered in underground engineering. The early version of the GSI system was presented as a table (Hoek et al., 1995) and a revised version was presented as a chart (Hoek and Brown, 1997). For good quality rocks, GSI value and RMR value are comparable. A classification system should be non-linear for poor rocks as strength deteriorates rapidly with weathering. Further, increased applications of computer modelling have created an urgent need for a classification system tuned to a computer simulation of rock structures.

3.3.4.1 Estimation of residual strength of rock masses

GSI system is extended for estimation of rock mass residual strength, Cai et al. (2007) proposed an adjustment of the original GSI value based on the two major controlling factors in the GSI system, block volume (V_b) and joint condition factor (J_C), to reach the residual values.

3.4 The use of classification in rock engineering

In comparison to many other civil engineering situations, the uncertainties in underground rock engineering are high. The design and different construction actions have to be based on:

a) The actual rock conditions encountered in the tunnel or underground opening during construction.
b) The geological model and assumed ground conditions from various types of investigations during the planning stage.

Pre-defined actions based on the use of classification systems have been shown to be an economic option in many cases. This is a very common situation in rock engineering and will be discussed further here. Some examples are:

Another example is when to reduce the length of the blast holes drilled to advance a tunnel. It is not very practical to use a continuous reduction; instead, classes are used like full round length, half round length or a quarter round lengths.

A common situation during the tunnelling process is to take the decision whether or not to use fore poling or spilling. This is a typical choice between two classes (alternatives).

CHAPTER – 4

SOME BASIC CONCEPTS

4.1 General

Classification of intact/jointed rock mass depends on the strength, material properties and other various parameters that directly/indirectly control their behaviour in nature. This study is an attempt to compare various classification system based on their strength for intact/jointed rock mass. Geotechnical tests were conducted to find out the strength of intact/jointed rock specimen. Based on the results, classification of intact/jointed rock mass has been done. Other than geotechnical studies, few other studies has been done in order to find out the material composition of the rock specimen. This study can be helpful in analysis of stability of rock masses such as tunnels, slopes, foundations, and excavatability of rock strata.

4.2 X-Ray Diffraction Analysis

X-Ray powder Diffraction analysis is a powerful method by which X-Rays of a known wavelength are passed through a sample to be identified in order to identify the crystal structure. The X-ray diffraction (XRD) test was used to determine the phase compositions of Plaster of Paris. The basic principles underlying the identification of minerals by XRD technique is that each crystalline substance has its own characteristics atomic structure which diffracts x-ray with a particular pattern. In general the diffraction peaks are recorded on output chart in terms of 2θ, where θ is the glancing angle of x-ray beam. The 2 θ values are then converted to lattice spacing „d" in angstrom unit using Bragg"s law, $d = \lambda/2n \sin\theta$; where n is an integer & λ = wave length of x-ray specific to target used. The X-Ray detector moves around the sample and measures the intensity of these peaks and the position of these peaks [diffraction angle 2θ].

4.3 SEM/EDX Analyses

A scanning electron microscope (SEM) is a type of electron microscope that produces images of a sample by scanning it with a focused beam of electrons. The electrons interact with electrons in the sample, producing various signals that can be detected and that contain information about the sample's surface topography and composition. The electron beam is generally scanned in a raster scan pattern, and the beam's position is combined with the detected signal to produce an image. SEM can achieve resolution better than 1 nanometre. Specimens can be observed in high vacuum, low vacuum and in environmental SEM specimens can be observed in wet conditions.

4.4 Uniaxial compressive strength

The uniaxial compressive strength of rock mass is represented in a non-dimensional form as the ratio of compressive strength of jointed rock and that of intact rock. The ratio of uniaxial compressive strength is expressed as below:

$$\sigma_{cr} = \sigma_{cj}/\sigma_{ci}$$

Where, σ_{cj} = uniaxial compressive strength of jointed rock

σ_{ci} = uniaxial strength of intact rock.

The uniaxial compressive strength of the experimental data should be plotted against the joint factor. The joint factor for the experimental specimen should be estimated based on the joint orientation, strength and spacing.

4.5 Elastic Modulus

Elastic modulus expressed as the tangent modulus at 50 % of stress failure is considered in this analysis. The elastic modulus ratio is expressed as

$$E_r = E_j / E_i$$

Where, E_j =is the tangent modulus of jointed rock; E_i =is the tangent modulus of intact rock.

Table 4.1 Values of inclination parameter (n) with respect to orientation angle (β°)

Orientation of joint (β°)	**Inclination parameter (n)**	**Orientation of joint (β°)**	**Inclination parameter (n)**
0	0.810	50	0.306
10	0.460	60	0.465
20	0.105	70	0.634
30	0.046	80	0.814
40	0.071	90	1.00

Table 4.2 Strength of jointed and intact rock mass

Class	Description	UCS , MPa
A	Very high strength	>250
B	High strength	100-250
C	Moderate strength	50-100
D	Medium strength	25-50
E	Low strength	5-25
F	Very low strength	<5

(After Ramamurthy and Arora, 1994)

Table 4.3 Modulus ratio classification of intact and jointed rocks

Class	Description	UCS , MPa
A	Very high modulus ratio	>500
B	High modulus ratio	200-500
C	Medium modulus ratio	100-200
D	Low modulus ratio	50-100
E	Very low modulus ratio	<50

(After Ramamurthy and Arora, 1994)

CHAPTER – 5

LABORATORY INVESTIGATION

5.1 General

In order to classify rock based on their strength and properties, various laboratory tests have been conducted. This chapter presents experimental investigation details which were carried to find out the shear strength and deformation properties of the rock joints. This chapter includes the details about materials used; preparation of specimens, curing, making joints in specimen, experimental set up, test procedure, and parameters studied.

5.2 Materials used

In this present study, plaster of Paris, plaster of Paris and Lime is used to simulate weak rock jointed mass, because of its ease in casting as well as it is flexible and it hardens instantly. Plaster of Paris can be used to simulate any kind of joint as required. It is observed that plaster of Paris has been used as model material to simulate weak rock mass in the field. Many researchers have used plaster of Paris because of its ease in casting, flexibility, instant hardening, low cost and easy availability. Various joint can be made by plaster of Paris. The reduced strength and deformed abilities in relation to actual rocks makes plaster of Paris, plaster of Paris and Lime one of the perfect materials for modelling in geotechnical engineering and hence it was used to prepare model for the present study.

5.3 Preparation of specimens

Plaster of Paris and Lime were procured from the local market. Number of trial tests was carried out for both the samples with different percentage of distilled water. Different specimens were tested for uniaxial compressive strength. The water content was determined as 30% for plaster of Paris specimen and 31% for Lime-POP mix specimen. A uniform paste was made in a bowl. Uniform mix was transferred into a mould in three layers .Care was taken that while transferring the mix into the mould, it was kept vibrating on a vibrating table for about two minutes. Vibration of mould was done in order to achieve proper compaction and thus making the specimen free from air voids. After that it was allowed to set and finally specimen was taken out of the mould manually with the help of an extruder. Same procedure has been repeated to prepare required number of test specimens. All the specimens were kept at room temperature for 48 hours.

The following standards have been suggested by I.S.R.M committee on Laboratory Test (1972) for compressive strength test:

- The ends of the specimen shall be flat to 0.02 mm (0.0008 in)
- The ends of the specimen shall be perpendicular to the axis of the specimen within .001 radian(3.5 minute)
- The number of specimens to be tested depends on the variability of the results and the desired accuracy and reliability of the mean value. Ten or more specimens are preferable to determine the strength of rocks.
- The sides of the specimen shall be smooth and free of abrupt irregularities and straight to within 0.3 mm (0.012 in) over the full length of the specimen.

5.4 Curing

After keeping the specimens in oven, they are placed inside desiccators containing a solution of concentrated sulphuric acid (47.7cc) mixed with distilled water (52.3cc). This is done mainly to maintain the relative humidity in range of 40% to 60%. Specimens are allowed to cure inside the desiccators till constant weight is obtained (about 15 days). Before testing each specimen of plaster of Paris obtaining constant weight dimensioned to L/D = 2:1, at L = 76 mm, D = 38 mm.

5.5 Making joints in specimens

For making rough joints, the following accessories were used:

1. Pencil
2. Scale
3. "V" block
4. Chisel
5. Protractor
6. Light weight hammer

On the surface of specimen two longitudinal lines were drawn opposite to each other. Protractor was used to mark the desired orientation angle with respect to the central longitudinal line. Then this marked specimen is placed on the "V" block and with the help of chisel keeping its edge along the drawn line, hammered continuously to break along the line. It is observed that the joints thus formed come under a category of rough joint. The uniaxial compressive strength test and direct shear test are conducted on intact specimens, jointed specimens with single and double joints.

5.6 Experimental setup and test procedure

In the present study, specimens were tested to obtain their uniaxial compressive strength, deformation behaviour and shear parameters. Uniaxial compression test, direct shear test was carried out in order to acquire these parameters as mentioned above. These tests were carried as per ISRM and ARE codes. On the prepared specimen of jointed rock mass uniaxial compression test was carried out (as per ASTM D2938)in order to obtain the ultimate compressive strength of jointed rock mass with respect to various orientation angles starting from 0° to 90°.

5.7 Uniaxial compressive strength

In Uniaxial compressive strength test the cylindrical specimens were subjected to major principal stress till the specimen fails due to shearing along a critical plane of failure. In this test the samples were fixed to cylindrical in shape, length 2 to 3 times the diameter; ends maintained flat within 0.02mm. Perpendicularity of the axis was not deviated by 0.001radian and the specimens were tested within 30days. The prepared specimens (L=76 mm, D=38 mm)

Calibration chart

Proving ring no - 1004

Capacity = 20 kN

1 Div. or LC = 24.242 N

Dial gauge least count= 0.01mm

5.8 Direct Shear test

The direct shear test was conducted to determine (roughness factor) joint strength r = tan ϕ_j in order to predict the joint factor J_f (Arora 1987). These test were carried out on conventional direct shear test apparatus (IS: 1129,1985) with certain modifications required for placement of specimens inside the box. Two identical wooden blocks of sizes 59X59X12 mm each having circular hole diameter of 39 mm at the Centre were inserted into two halves of shear box the specimen is then place inside the shear box (60 x 60 mm). The cylindrical specimen broken into the two equal parts was fitted into the circular hole of the wooden blocks, so that the broken surface match together and laid on the place of shear i.e. the Contact surface of two halves of the shear box. Values of shear stress for different values of normal stress of intact specimen of plaster of Paris and Lime-POP mix in direct shear test.

Direct shear test calibration chart

Proving ring No - 099

1 Div. or LC = 3.83 kN

Capacity = 2.5 kN

5.9 Parameters studied

The main objective of the experimental investigation was to study the following aspects

- Strength classification
- To make a classification of jointed rock masses
- UCS test of the intact and jointed specimen.
- Joint factor with respect to various orientation angles
- Compressive strength ratio and elastic modulus ratio
- Failure strain classification

Uniaxial compressive strength of specimens was conducted in order to determine the strength as well as the deformation characteristic of intact and jointed specimens with single and double joint. The specimens were tested for different orientation angles such as 10, 20, 30, 40, 50, 60, 70, 80, 90 degrees and for intact specimens. The jointed specimens were placed inside a rubber membrane before testing of U.C.S to avoid slippage along the joints just after application of the load. Some of these specimens are shown in Fig 5.1.

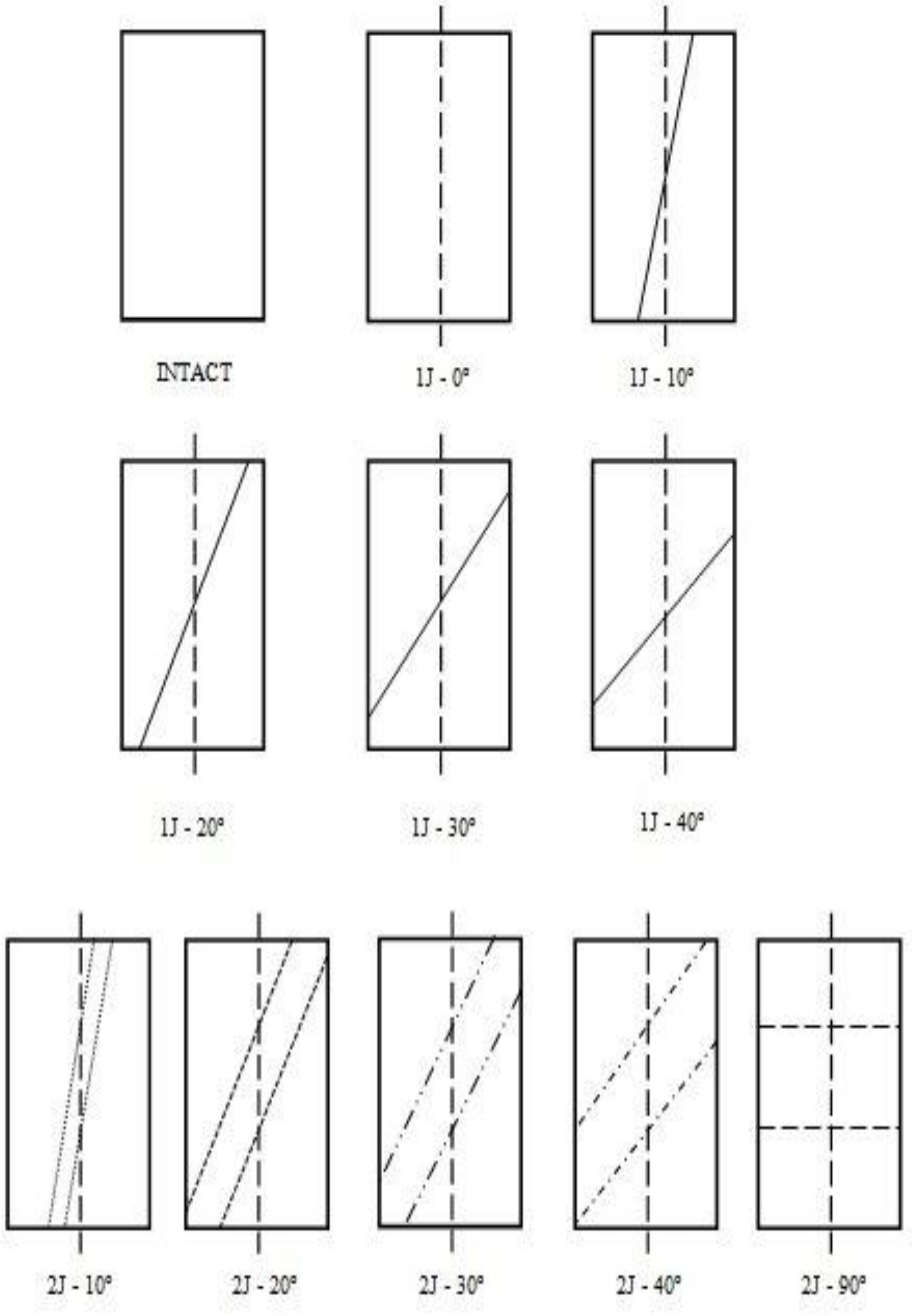


Fig 5.1 Types of joints studied in plaster of Paris and lime-plaster of Paris mix specimens. (Single and double jointed specimens are shown here)

5.10 Types of joints studied

Table 5.1 Types of joint studied for uniaxial compressive strength

Types of joints with major principal axis for single joint specimens	1J-0º	IJ-10º	1J-20º	1J-30º	1J-40º	1J-50º	1J-60º	1J-70º	1J-80º	91J-0º
Types of joints with major principal axis for double joint specimens	-	2J-10º	2J-20º	2J-30º	2J-40º	2J-50º	2J-60º	2J-70º	2J-80º	2J-90º

CHAPTER - 6

RESULT AND DISCUSSION

6.1 Results from SEM/EDX, XRD

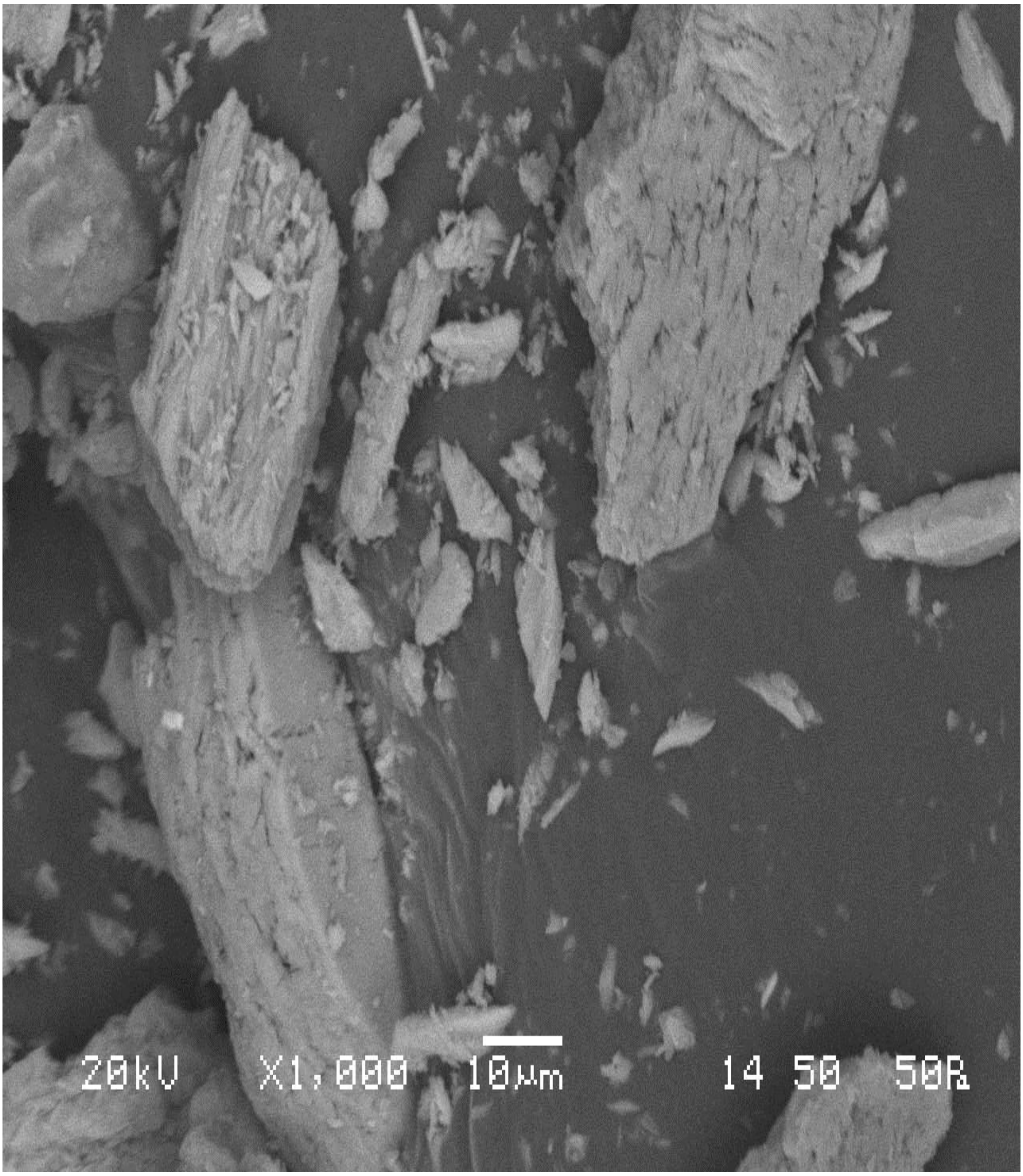


Fig 6.1 Microstructure of plaster of Paris at X1000 and 10µm

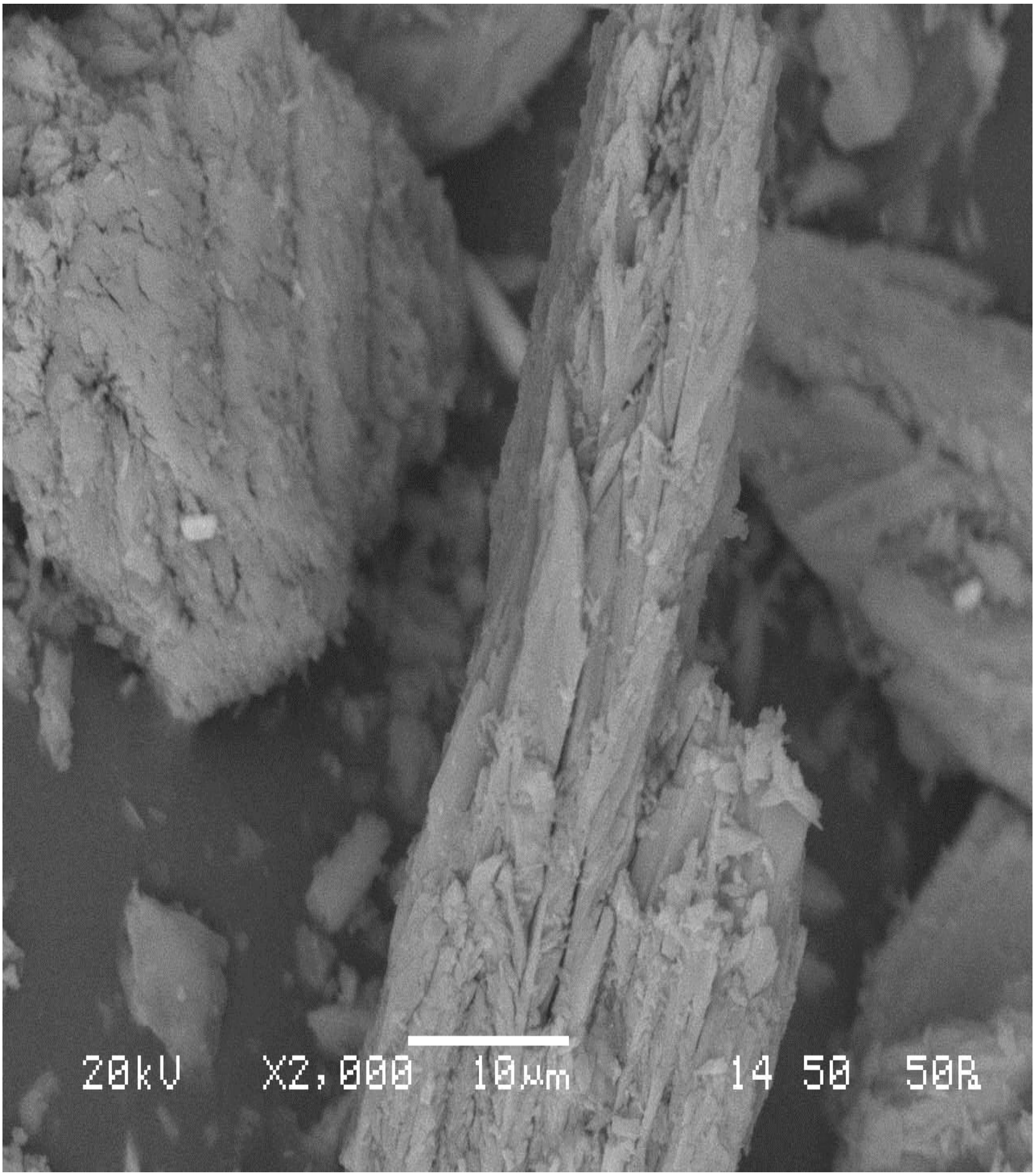


Fig 6.2 Microstructure of plaster of Paris at X2000 and 10μm

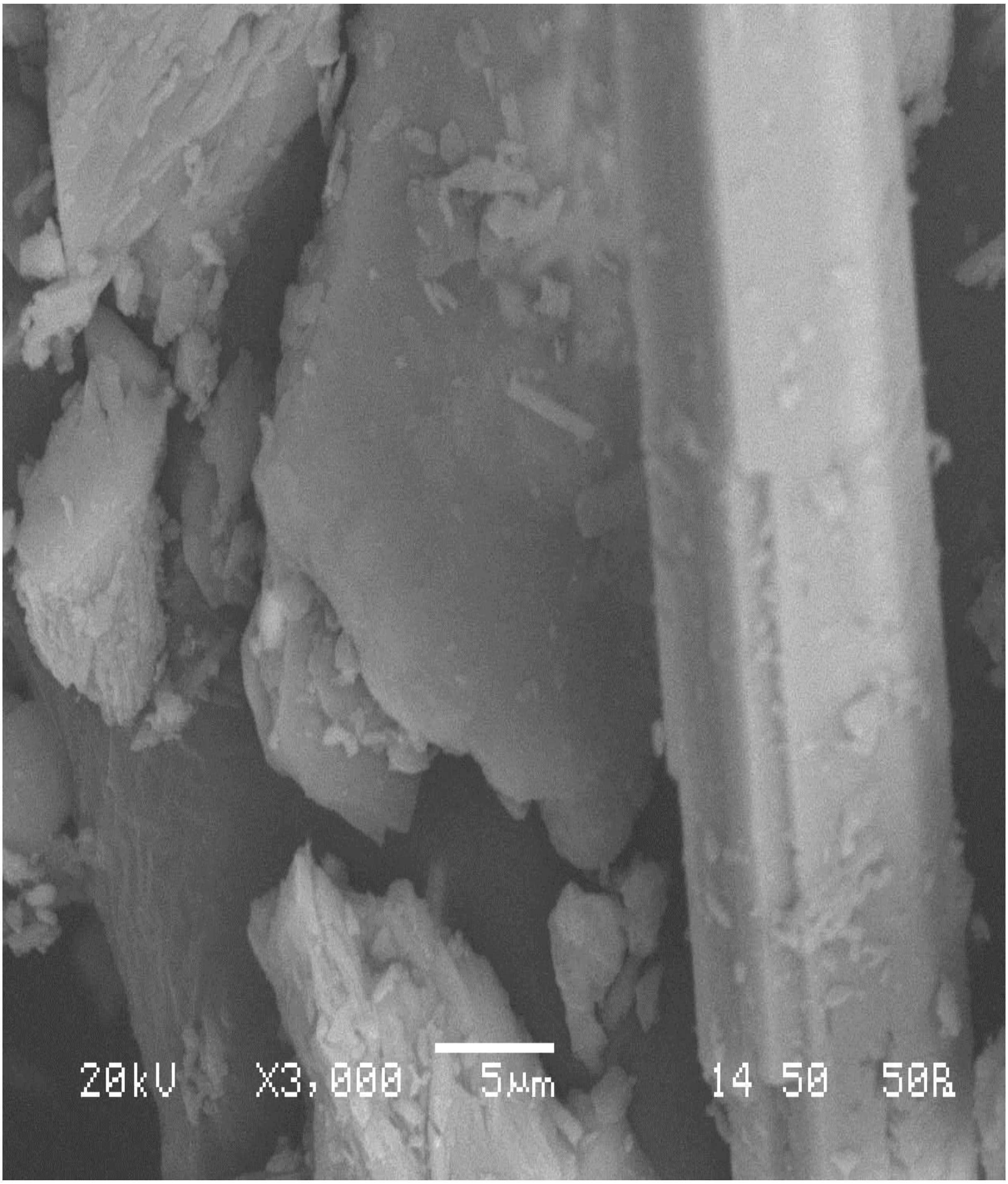
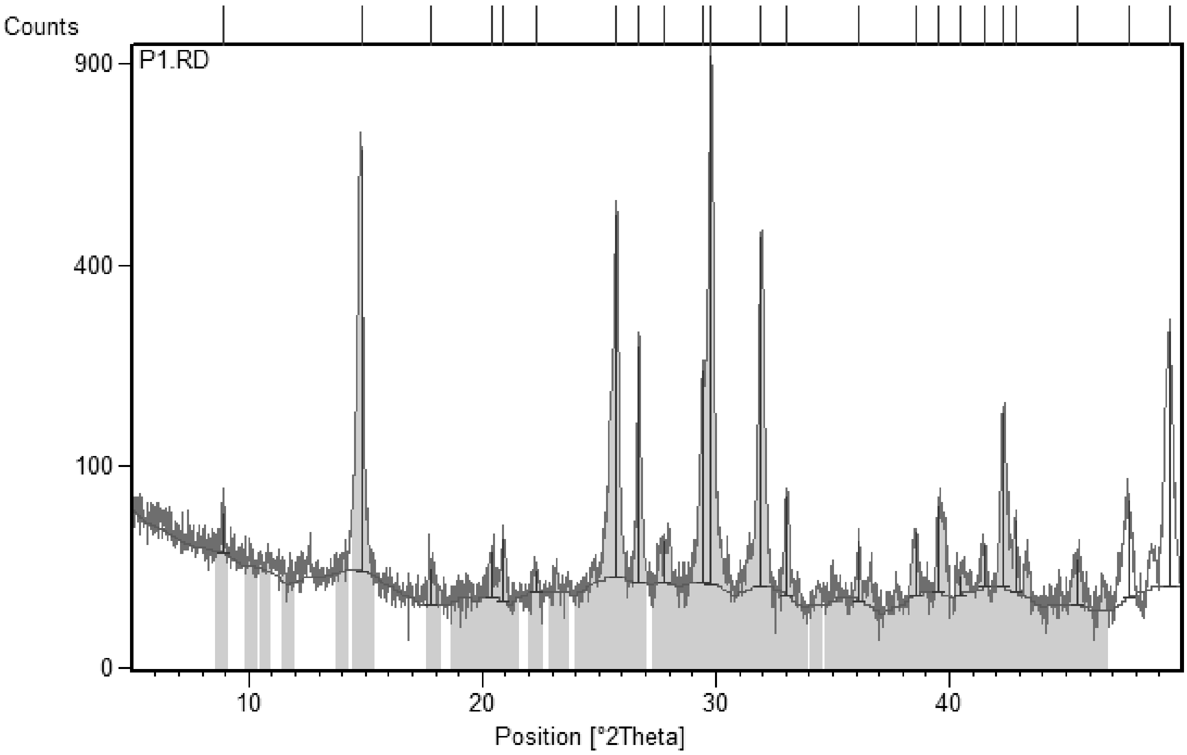


Fig 6.3 Microstructure of plaster of Paris at X3000 and 5µm



Accepted: Ref. Pattern: 83-0438 Scale factor: 0.819

No.	Visib..	Ref. Code	Score	Compound Name	Chemical Formula	Scale Factor
1	✓	83-0438	69	Calcium Sulfate Hydrate	Ca (S O4) (H2 O)0.5	0.819
2	✓	32-0415	43	Graphite Hydrogen Nitrate	C16 H N O3	0.138
3	✓	29-0127	24	Antimony Fluoride Iodide	Sb I F12	0.230
4	✓	82-1562	27	Silicon Oxide	Si O2	0.381
5	✓	31-0261	23	Scawtite	Ca7 (Si6 O18) (C O3	0.311
6	✓	24-0027	28	Calcite	Ca C O3	0.188
7	✓	17-0912	27	Calcium Oxide	Ca O	0.381

Position [º2 Theta]

Fig 6.4 Microscopic pattern of plaster of Paris

6.2 Direct shear test results of plaster of Paris test specimen

The roughness parameter (r) which is the tangent value of the friction angle (Φ_j) was obtained from the direct shear test conducted at different normal stresses. The value of cohesion (C_j) for jointed specimens of plaster of Paris has been found as 0.178 MPa and value of friction angle (Φ_j) found as 39º. Hence the roughness parameter ($r = \tan\Phi_j$) comes to be 0.809 for the specimens of plaster of Paris tested. Cross sectional area of samples = 1134mm^2.

Table 6.1 Values of shear stress for different values of normal stress on jointed specimens of plaster of Paris in direct shear stress test.

Normal stress ,σ_n (MPa)	**Shear stress, τ (MPa)**
0.049	0.298
0.098	0.417
0.147	0.537

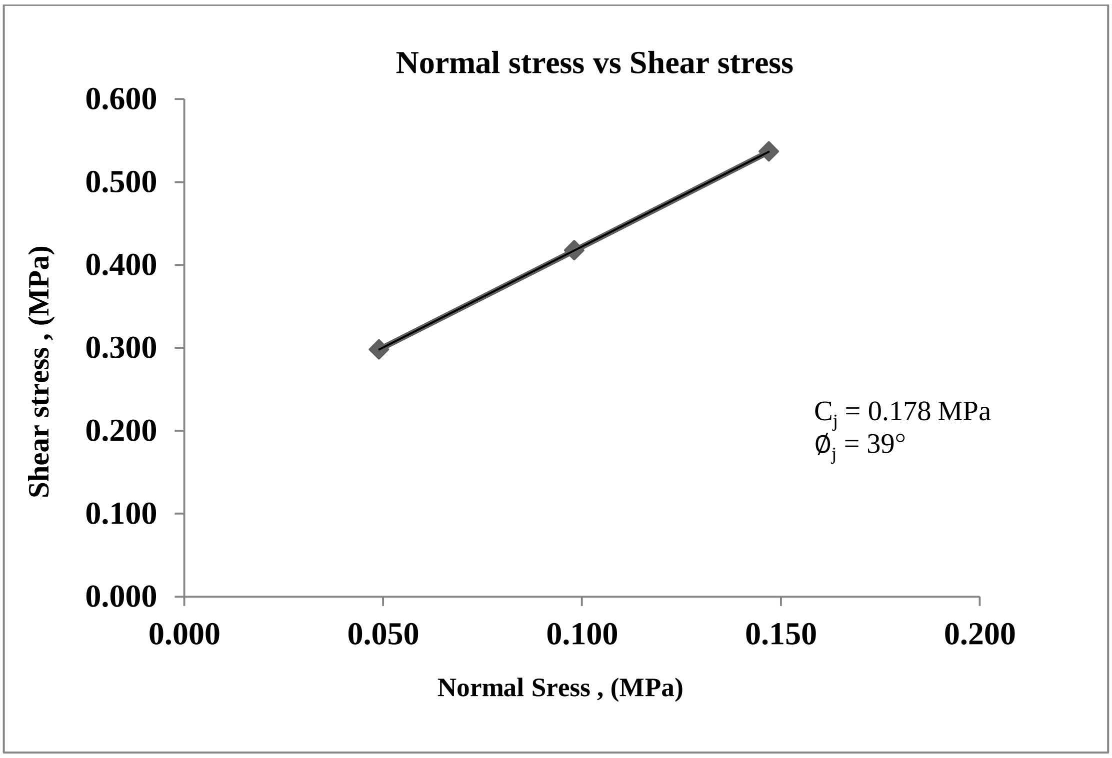


Fig 6.5 Normal stress vs. Shear stress of plaster of Paris jointed specimen

6.3 Uniaxial compression test results of plaster of Paris intact specimen

The variations of the stress with strain as obtained by uniaxial compression strength test for the intact specimen of plaster of Paris is and its corresponding stress vs. strain values are presented in Table no.6.2 The value of uniaxial compression strength (σ_{ci}) evaluated from the above tests was found to be 9.62 MPa. The modulus of elasticity of intact specimen (E_{ti}) has been calculated at 50% of the σ_{ci} value to account the tangent modulus. The value of E_{ti} was found as 361.17 MPa.

POP intact specimen details for UCS test

Length of specimen = 76mm

Diameter of specimen = 38mm

Cross sectional area of the specimen = 1134 Sqmm

Table 6.2 Values of stress and strain for intact plaster of Paris specimen

Axial strain, ε_a(%)	**Uniaxial compressive strength, σ_{ci} (MPa)**
0	0
0.658	2.03
1.316	3.96
1.974	5.45
2.632	7.81
3.289	9.41
3.947	9.62
4.605	9.52

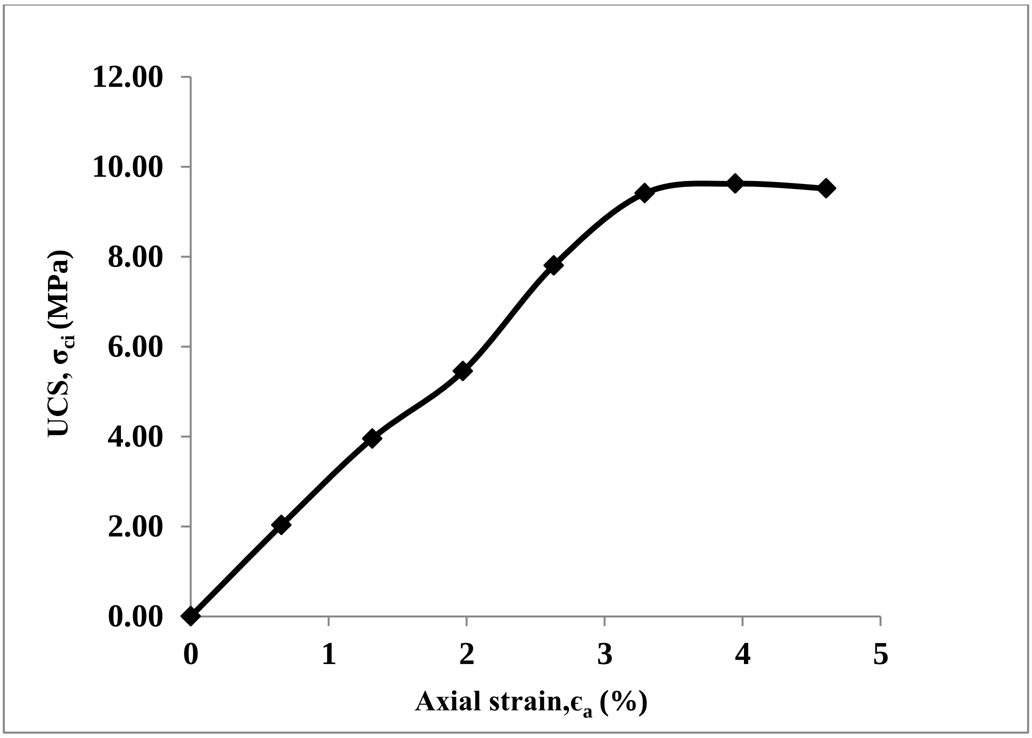


Fig 6.6 Axial strain vs. Stress for uniaxial compressive strength of plaster of Paris intact specimen

Table 6.3 Physical and engineering properties of plaster of Paris obtained from the test

Sl No.	Property/Parameter	Values
1	Uniaxial compressive strength, σ_{ci} (MPa)	9.62
2	Tangent modulus, (E_{ti}) (MPa)	361.17
3	Cohesion intercept, c_j (MPa)	0.178
4	Angle of friction, Φ_j (degree)	39

6.4 Experiment conducted for jointed specimen of plaster of Paris and Lime-plaster of Paris mix

The Uniaxial compressive strength of intact specimens obtained from the test results has already been found out. In similar manner, the uniaxial compressive strength (σ_{cj}) as well as modulus of elasticity (E_{tj}) for the jointed specimens was evaluated after testing the jointed specimens. In this case, the jointed specimens are placed inside a rubber membrane before testing, to avoid slippage along the critical joints. After obtaining the values of (σ_{cj}) and E_{ti} for different orientations (β°) of joints, it was observed that the jointed specimens exhibit minimum strength when the joint orientation angle was at 30° and maximum when angle was 90° The values of (σ_{cr}) for different orientation angle (β°) were obtained with the help of the following relationship:

$$\sigma_{cr} = \sigma_{cj}/\sigma_{ci} \tag{6.1}$$

The values of joint factor (J_f) were evaluated by using the relationship:

$$J_f = J_n / (n*r) \tag{6.2}$$

Arora (1987) has suggested the following relationship between J_f and σ_{cr} as,

$$\sigma_{cr} = e^{-0.008*Jf} \tag{6.3}$$

Arora (1987) has suggested the following relationship between J_f and E_r as,

$$\mathrm{Er} = \mathrm{e}^{-1.15*10^{-2}*J_f} \tag{6.4}$$

Padhy (2005) has suggested the following relationship between J_f and σ_{cr} as,

$$\sigma_{cr} = e^{-0.09*Jf} \tag{6.5}$$

Padhy (2005) has suggested the following relationship between J_f and E_r as,

$$E_r = \mathrm{e}^{-1.25*10^{-2}*J_f} \tag{6.6}$$

Table 6.4 Values of J_n, J_f, σ_{cj}, σ_{cr} for plaster of Paris jointed specimens (single joint)

Joint type in degrees	J_n	n	r = tanΦj	J_f = Jn/(n*r)	σcj (MPa)	$\sigma_{cr} = \sigma_{cj}/\sigma_{ci}$	Predicted Arora(1987) $\sigma_{cr} = e^{-0.008*Jf}$	Predicted Padhy(2005) $\sigma_{cr} = e^{-0.09*Jf}$
0	13	0.810	0.809	19.839	8.770	0.9116	0.85325	0.16772
10	13	0.460	0.809	34.933	7.810	0.8119	0.75619	0.04311
20	13	0.105	0.809	153.040	4.170	0.4335	0.29396	0.00000
30	13	0.046	0.809	349.331	1.920	0.1996	0.06114	0.00000
40	13	0.071	0.809	226.327	3.310	0.3441	0.16355	0.00000
50	13	0.306	0.809	52.514	6.420	0.6674	0.65697	0.00886
60	13	0.465	0.809	34.557	7.060	0.7339	0.75846	0.04459
70	13	0.634	0.809	25.346	7.380	0.7672	0.81647	0.10217
80	13	0.814	0.809	19.741	7.590	0.7890	0.85391	0.16920
90	13	1.000	0.809	16.069	8.660	0.9116	0.87937	0.23546

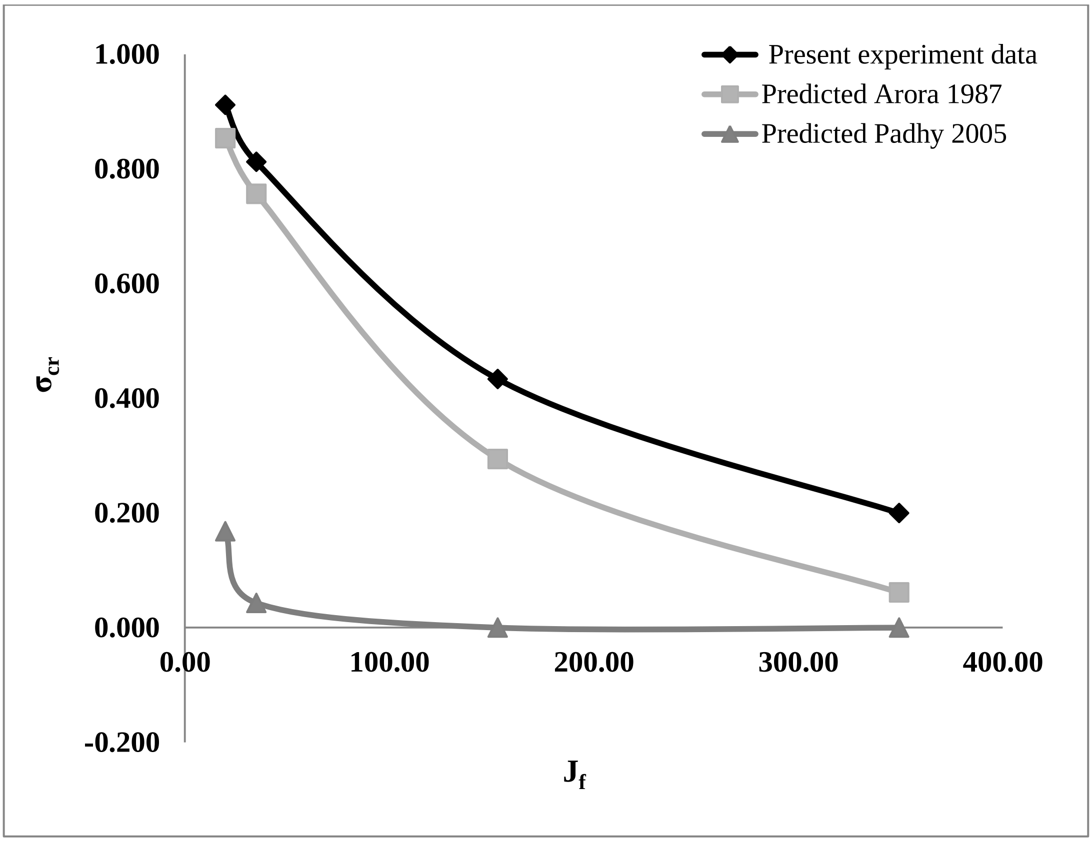


Fig 6.7 Joint factor vs. Compressive strength ratio (POP Single joint specimen)

Table 6.5 Values of J_n, J_f, σ_{cj}, σ_{cr} for plaster of Paris jointed specimens (double joint)

Joint type in degrees	**J_n**	**n**	**r = tanΦj**	**J_f = Jn/(n∗r)**	**σ_{cj} (MPa)**	**$\sigma_{cr} = \sigma_{cj}/\sigma_{ci}$**	**Predicted Arora(1987) $\sigma_{cr} = e^{-0.008*Jf}$**	**Predicted Padhy(2005) $\sigma_{cr} = e^{-0.09*Jf}$**
10	26	0.460	0.809	69.866	6.630	0.6892	0.57182	0.00186
20	26	0.105	0.809	306.080	2.250	0.2339	0.08641	0.00000
30	26	0.046	0.809	698.662	0.640	0.0665	0.00374	0.00000
40	26	0.071	0.809	452.654	1.710	0.1778	0.02675	0.00000
50	26	0.306	0.809	105.028	4.810	0.5000	0.43162	0.00008
60	26	0.465	0.809	69.115	5.670	0.5894	0.57527	0.00199
70	26	0.634	0.809	50.692	6.840	0.7110	0.66662	0.01044
80	26	0.814	0.809	39.482	7.060	0.7339	0.72916	0.02863
90	26	1.000	0.809	32.138	7.810	0.8119	0.77329	0.05544

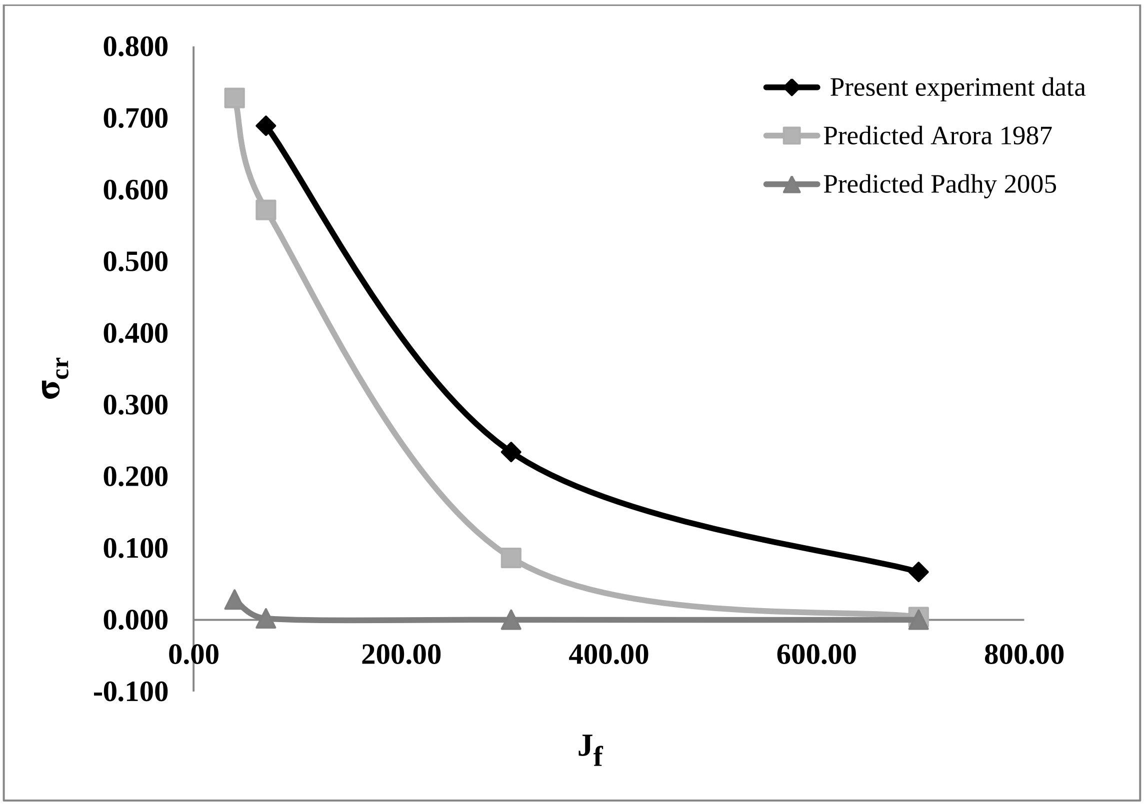


Fig 6.8 Joint factor vs. Compressive strength ratio (POP double joint specimen)

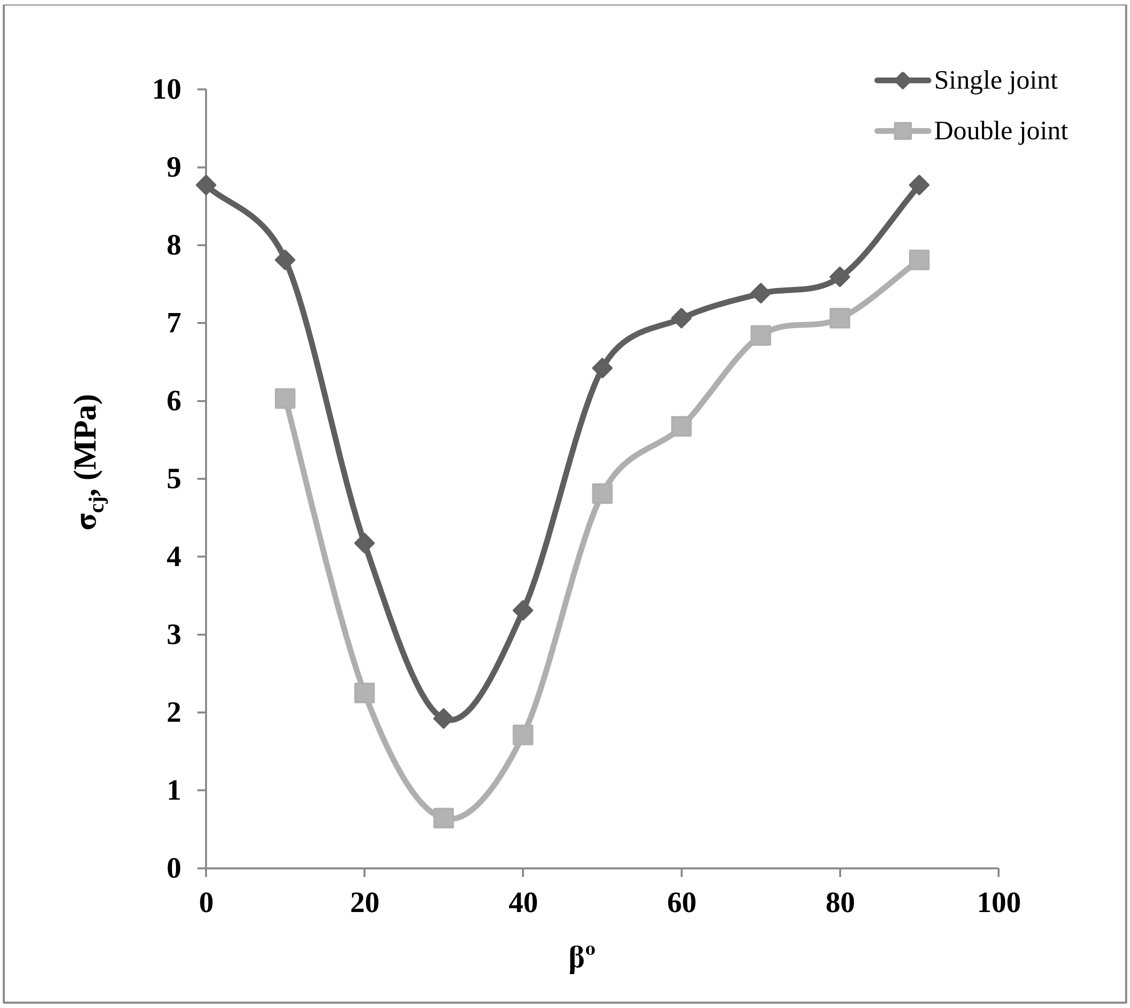


Fig 6.9 Orientation angle (β˚) vs. Uniaxial compressive strength, σ_{cj} (MPa) of plaster of Paris specimen represents the nature of compressive strength anisotropy

Table 6.6 Values of E_{tj}, E_r for plaster of Paris jointed specimens (single joint)

Joint type in degrees	J_n	n	r = tanΦj	J_f = Jn/(n*r)	E_{tj} (MPa)	Er =Etj/Eti	Predicted Arora(1987) $Er = e^{-1.15*10^{-2}*J_f}$	Predicted Padhy(2005) $E_r = e^{-1.25*10^{-2}*J_f}$
0	13	0.810	0.809	19.839	312.90	0.866	0.7960	0.7804
10	13	0.460	0.809	34.933	308.30	0.854	0.6692	0.6462
20	13	0.105	0.809	153.040	104.50	0.289	0.1721	0.1476
30	13	0.046	0.809	349.331	29.94	0.083	0.0180	0.0127
40	13	0.071	0.809	226.327	73.82	0.204	0.0741	0.0591
50	13	0.306	0.809	52.514	203.68	0.564	0.5467	0.5187
60	13	0.465	0.809	34.557	239.65	0.664	0.6721	0.6492
70	13	0.634	0.809	25.346	234.67	0.650	0.7472	0.7285
80	13	0.814	0.809	19.741	263.86	0.731	0.7969	0.7813
90	13	1.000	0.809	16.069	312.96	0.867	0.8313	0.8180

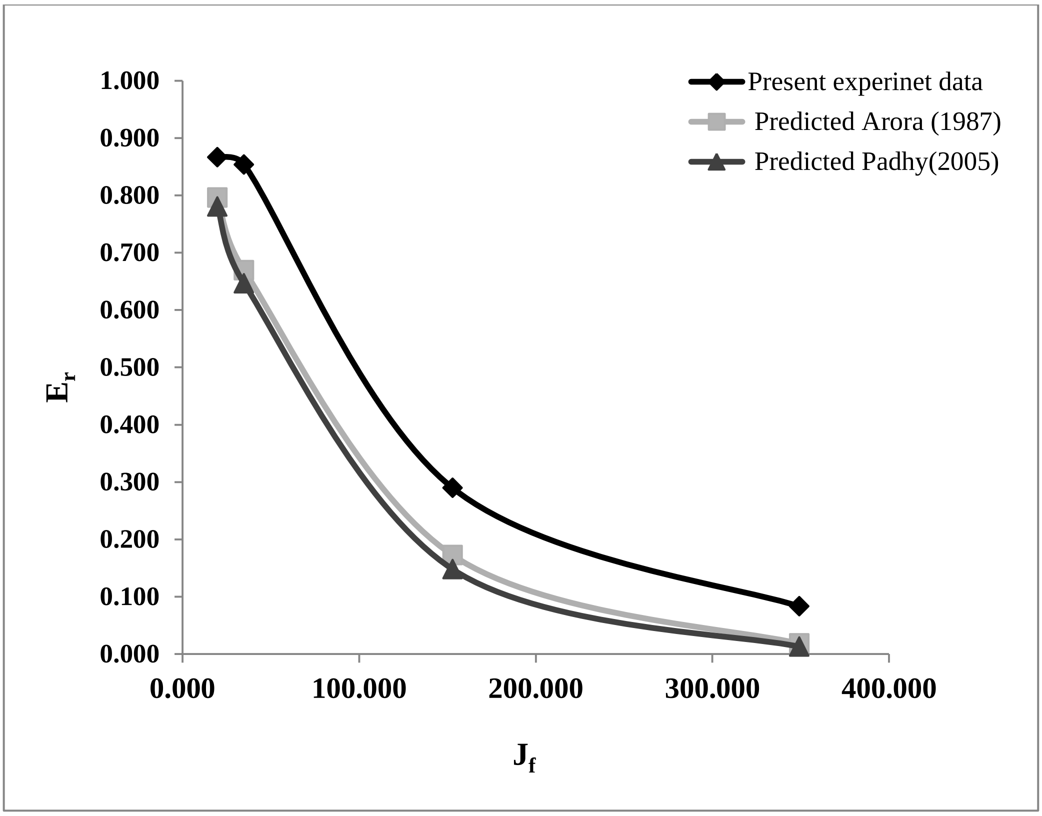


Fig 6.10 Joint factor vs. Modular ratio (POP single joint specimen)

Table 6.7 Values of E_{tj}, E_r for plaster of Paris jointed specimens (double joint)

Joint type in degrees	**J_n**	**n**	**r = tanΦj**	**J_f = Jn/(n*r)**	**E_{tj} (MPa)**	**Er =Etj/Eti**	**Predicted Arora(1987) Er = $e^{-1.15*10^{-2}*J_f}$**	**Predicted Padhy(2005) E_r = $e^{-1.25*10^{-2}*J_f}$**
10	26	0.460	0.809	69.866	225.00	0.623	0.4478	0.4176
20	26	0.105	0.809	306.080	63.00	0.174	0.0296	0.0218
30	26	0.046	0.809	698.662	18.46	0.051	0.0003	0.0002
40	26	0.071	0.809	452.654	27.44	0.076	0.0055	0.0035
50	26	0.306	0.809	105.028	96.73	0.268	0.2988	0.2691
60	26	0.465	0.809	69.115	171.13	0.474	0.4517	0.4215
70	26	0.634	0.809	50.692	196.00	0.543	0.5582	0.5307
80	26	0.814	0.809	39.482	199.00	0.551	0.6351	0.6105
90	26	1.000	0.809	32.138	269.00	0.745	0.6910	0.6692

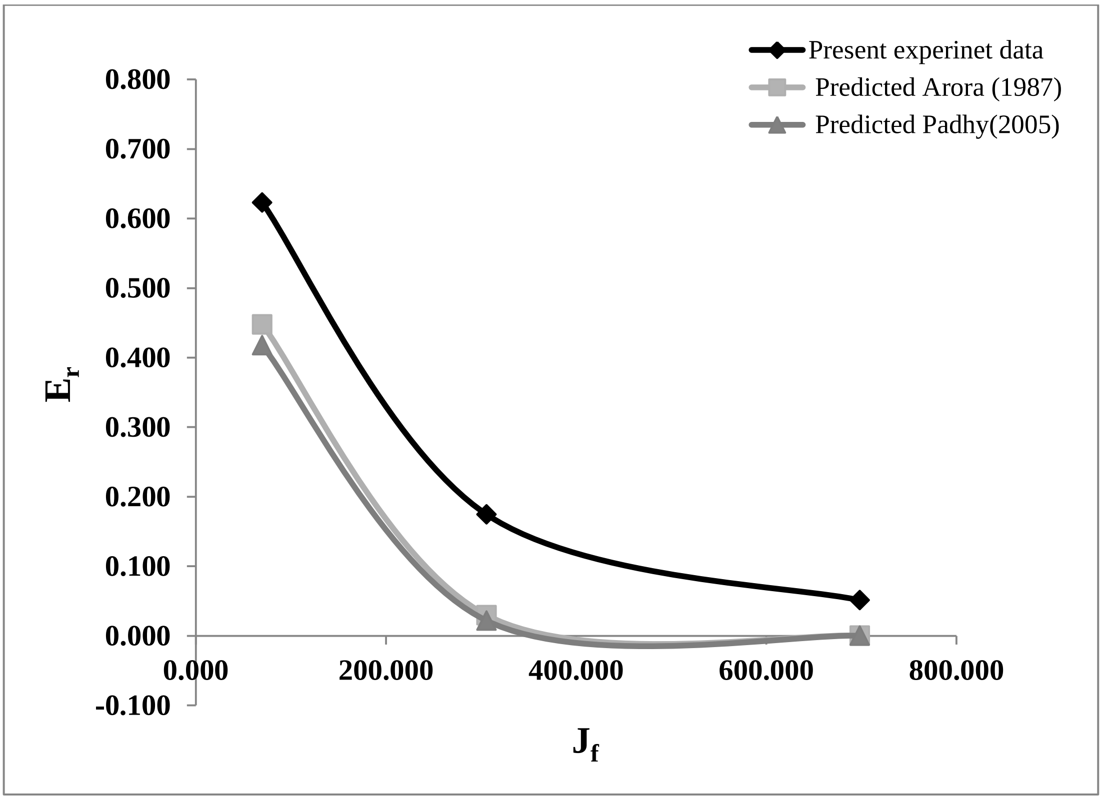


Fig 6.11 Joint factor vs. Modular ratio (POP double joint specimen)

6.5 Direct shear test results of Lime-POP mix test specimen

The roughness parameter (r) which is the tangent value of the friction angle (Φ_j) was obtained from the direct shear test conducted at different normal stresses. The value of cohesion (C_j) for jointed specimens of Lime- plaster of Paris mix specimen has been found as 0.169 MPa and value of friction angle (Φ_j) found as 38º. Hence the roughness parameter (r = tanΦ_j) comes to be 0.781 for the specimens of plaster of Paris tested.

Table 6.8 Values of shear stress for different values of normal stress for Lime-POP mix jointed specimen in direct shear stress test.

Normal stress ,σ_n (MPa)	**Shear stress, τ (MPa)**
0.049	0.253
0.098	0.387
0.147	0.507

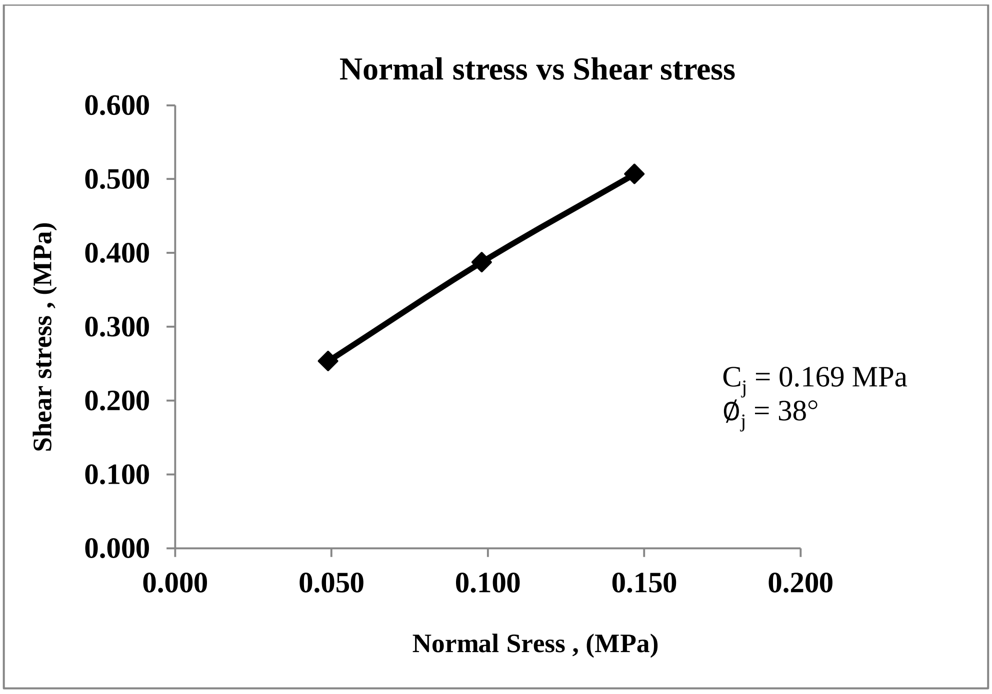


Fig 6.12 Normal stress vs. Shear stress of Lime-POP mix jointed specimen

6.6 Uniaxial compression test results of Lime-POP mix intact specimen

The variations of the stress with strain as obtained by uniaxial compression strength test for the intact specimen of Lime and plaster of Paris mix and its corresponding stress vs. strain values are presented in Table 6.9. The value of uniaxial compression strength (σ_{ci}) evaluated from the above tests was found to be 8.77 MPa. The modulus of elasticity of intact specimen (E_{ti}) has been calculated at 50% of the σ_{ci} value to account the tangent modulus. The value of E_{ti} was found as 274.03 MPa.

Lime-POP mix intact specimen details for UCS test

Length of specimen = 76mm

Diameter of specimen = 38mm

Cross sectional area of the specimen = 1134 Sqmm

Table 6.9 Values of stress and strain of Lime-POP mix intact specimen

Axial strain, ε_a (%)	**Uniaxial compressive strength, σ_{ci} (MPa)**
0	0
0.658	1.60
1.316	2.57
1.974	3.96
2.632	5.24
3.289	6.84
3.947	8.77
4.605	8.66

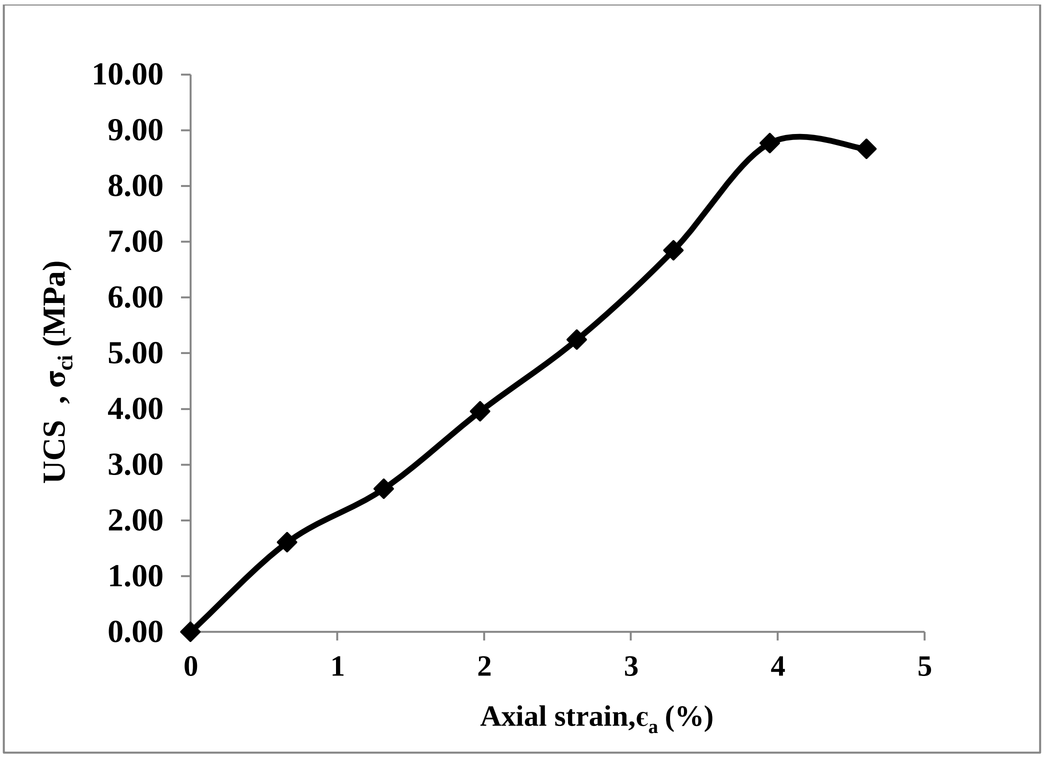


Fig 6.13 Axial strain vs. Stress for uniaxial compressive strength of Lime-POP mix intact specimen

Table 6.10 Physical and engineering properties of Lime-POP mix specimen obtained from the test

Sl No.	Property/Parameter	Values
1	Uniaxial compressive strength, σ_{ci} (MPa)	8.77
2	Tangent modulus, (E_{ti}) (MPa)	274.03
3	Cohesion intercept, c_j (MPa)	0.169
4	Angle of friction, Φ_j (degree)	38

Table 6.11 Values of J_n, J_f, σ_{cj}, σ_{cr} for Lime-POP mix single joint specimens

Joint type in degrees	J_n	**n**	**r =tanΦj**	$J_f = Jn/(n*r)$	**σcj (MPa)**	$\sigma_{cr} = \sigma_{cj}/\sigma_{ci}$	**Predicted Arora(1987)** $\sigma_{cr} = e^{-0.008*Jf}$	**Predicted Padhy(2005)** $\sigma_{cr} = e^{-0.09*Jf}$
0	13	0.810	0.781	20.550	6.840	0.780	0.84840	0.15732
10	13	0.460	0.781	36.185	5.100	0.582	0.74865	0.03852
20	13	0.105	0.781	158.527	2.890	0.330	0.28133	0.00000
30	13	0.046	0.781	361.855	1.600	0.182	0.05531	0.00000
40	13	0.071	0.781	234.441	2.460	0.281	0.15327	0.00000
50	13	0.306	0.781	54.396	5.030	0.574	0.64715	0.00748
60	13	0.465	0.781	35.796	6.310	0.719	0.75098	0.03989
70	13	0.634	0.781	26.254	7.060	0.805	0.81056	0.09415
80	13	0.814	0.781	20.449	7.810	0.891	0.84909	0.15876
90	13	1.000	0.781	16.645	8.050	0.918	0.87532	0.22356

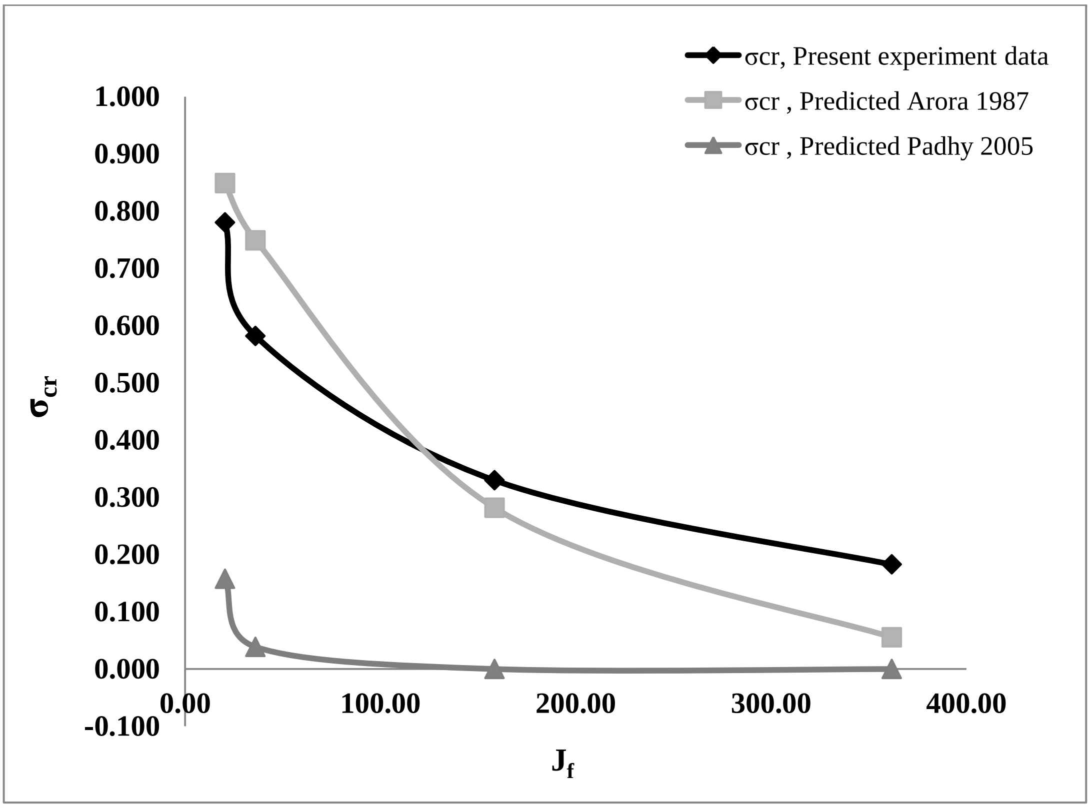


Fig 6.14 Joint factor vs. Compressive strength ratio for Lime-POP mix single joint specimen

Table 6.12 Values of J_n, J_f, σ_{cj}, σ_{cr} for Lime-POP mix double joint specimens

Joint type in degrees	**J_n**	**n**	**r = tanΦj**	**J_f = Jn/(n∗r)**	**σcj (MPa)**	**σ_{cr} = σ_{cj}/ σ_{ci}**	**Predicted Arora(1987) $\sigma_{cr} = e^{-0.008*Jf}$**	**Predicted Padhy(2005) $\sigma_{cr} = e^{-0.09*Jf}$**
10	26	0.460	0.781	72.371	4.010	0.457	0.56048	0.00148
20	26	0.105	0.781	317.054	1.980	0.226	0.07915	0.00000
30	26	0.046	0.781	723.710	1.020	0.116	0.00306	0.00000
40	26	0.071	0.781	468.882	1.800	0.205	0.02349	0.00000
50	26	0.306	0.781	108.793	4.120	0.470	0.41881	0.00006
60	26	0.465	0.781	71.593	5.250	0.599	0.56398	0.00159
70	26	0.634	0.781	52.509	6.040	0.689	0.65700	0.00886
80	26	0.814	0.781	40.898	6.980	0.796	0.72095	0.02520
90	26	1.000	0.781	33.291	7.120	0.812	0.76619	0.04998

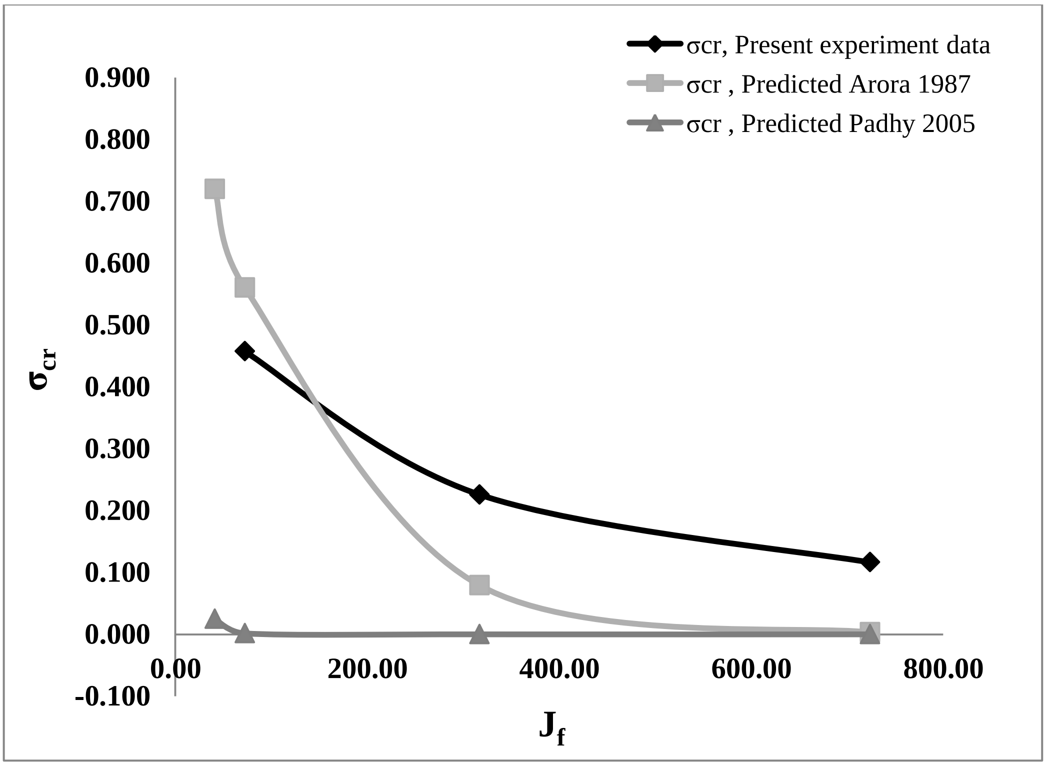


Fig 6.15 Joint factor vs. Compressive strength ratio for Lime-POP mix double joint specimen

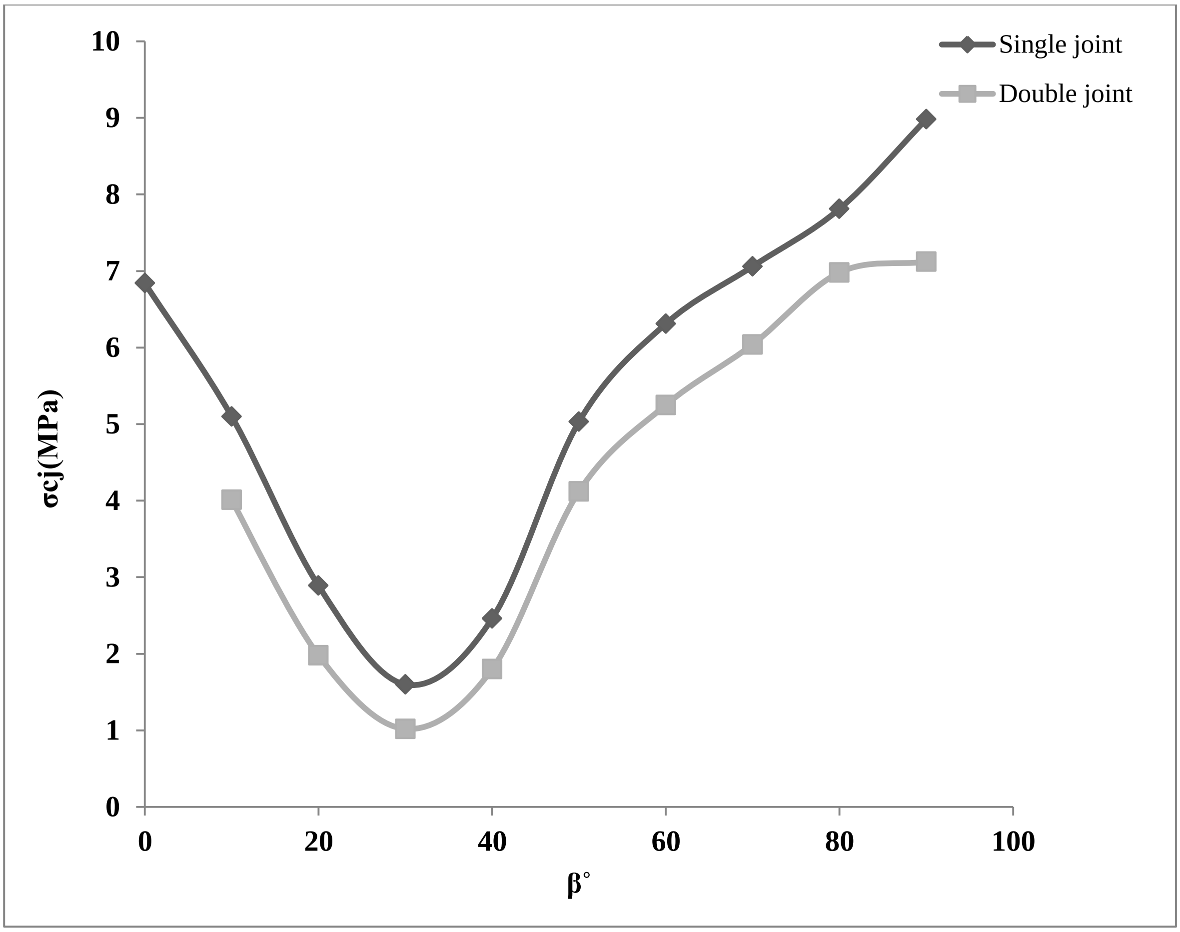


Fig 6.16 Orientation angle (β˚) vs. Uniaxial compressive strength, σ_{cj} (MPa) of Lime-POP mix specimen represents the nature of compressive strength anisotropy

Table 6.13 Values of E_{tj}, E_r for Lime-POP mix single joint specimens

Joint type in degrees	Jn	n	r = tanΦj	J_f = Jn/(n*r)	Etj (MPa)	Er =Etj/Eti	Predicted Arora(1987) $Er = e^{-1.15*10^{-2}*J_f}$	Predicted Padhy(2005) $E_r = e^{-1.25*10^{-2}*J_f}$
0	13	0.810	0.781	20.550	216.04	0.788	0.790	0.773
10	13	0.460	0.781	36.185	161.08	0.588	0.660	0.636
20	13	0.105	0.781	158.527	90.29	0.329	0.162	0.138
30	13	0.046	0.781	361.855	49.98	0.182	0.016	0.011
40	13	0.071	0.781	234.441	77.69	0.284	0.067	0.053
50	13	0.306	0.781	54.396	158.87	0.580	0.535	0.507
60	13	0.465	0.781	35.796	199.31	0.727	0.663	0.639
70	13	0.634	0.781	26.254	220.57	0.805	0.739	0.720
80	13	0.814	0.781	20.449	244.00	0.890	0.790	0.774
90	13	1.000	0.781	16.645	251.00	0.916	0.826	0.812

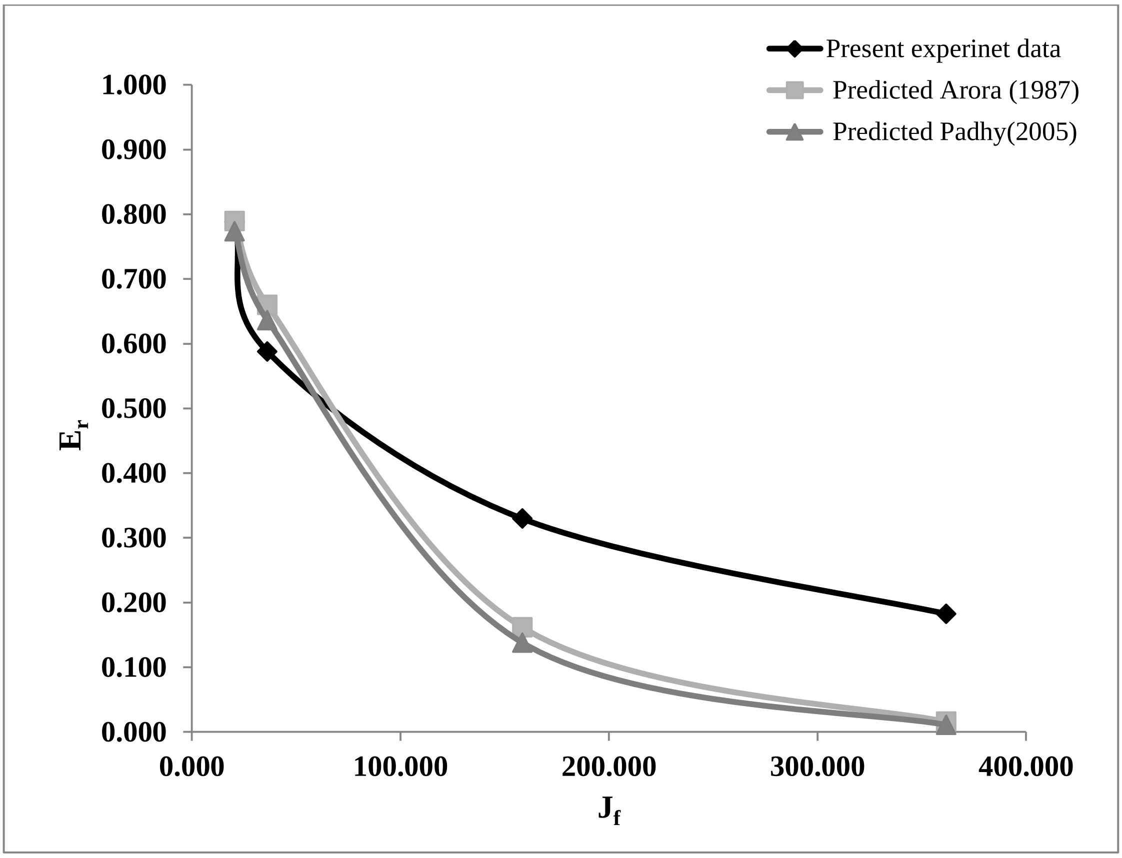


Fig 6.17 Joint factor vs. Modular ratio for Lime-POP mix single joint specimen

Table 6.14 Values of E_{tj}, E_r for Lime-POP mix double joint specimens

Joint type in degrees	Jn	n	r = tanΦj	$J_f = Jn/(n*r)$	Etj (MPa)	$E_r = E_{tj}/E_{ti}$	Predicted Arora(1987) $Er = e^{-1.15*10^{-2}*J_f}$	Predicted Padhy(2005) $E_r = e^{-1.25*10^{-2}*J_f}$
10	26	0.460	0.781	72.371	126.59	0.462	0.435	0.405
20	26	0.105	0.781	317.054	61.85	0.226	0.026	0.019
30	26	0.046	0.781	723.710	31.86	0.116	0.000	0.000
40	26	0.071	0.781	468.882	56.34	0.206	0.005	0.003
50	26	0.306	0.781	108.793	129.41	0.472	0.286	0.257
60	26	0.465	0.781	71.593	170.24	0.621	0.439	0.409
70	26	0.634	0.781	52.509	196.61	0.717	0.547	0.519
80	26	0.814	0.781	40.898	218.06	0.796	0.625	0.600
90	26	1.000	0.781	33.291	223.04	0.814	0.682	0.660

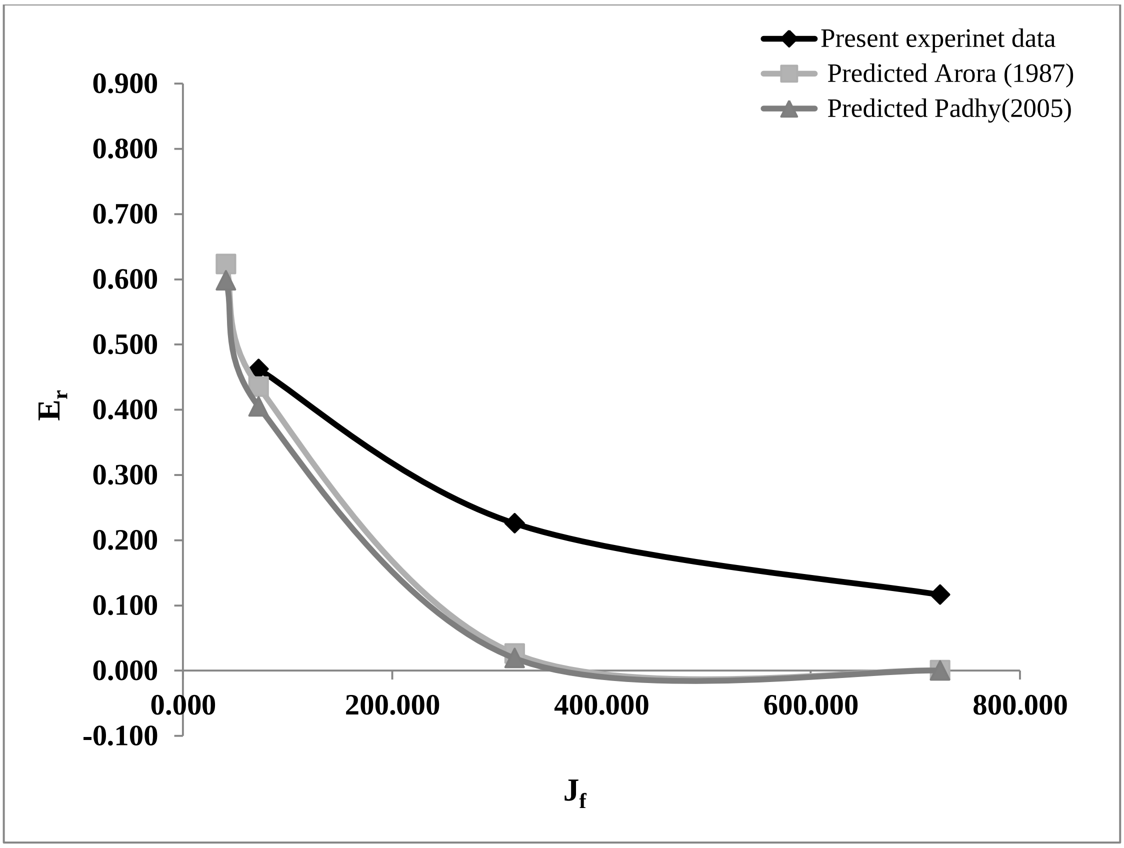


Fig 6.18 Joint factor vs. Modular ratio for Lime-POP mix double joint specimen

6.7 Classification of intact rock

The classification proposed by Deere and Miller (1966) for intact rocks is based on the combined influence of the uniaxial compressive strength (σ_{ci}), and tangent modulus (E_{ti}) at 50% of the failure stress. This approach has been widely recognized as a realistic and useful engineering classification which takes into account more than one measurable property at a time. Based on these properties, they categorized rocks into a number of classes assigning two lettered combination; the first letter refers to the compressive strength range and the second letter refers to the modulus ratio. The limits of the various classes of the intact rocks are classified the effect of the seepage pressure or confining pressure is not considered.

6.7.1 Classification of intact plaster of Paris specimen

Table 6.15 A Strength classification of intact rock after Deere and Miller, 1966

Class	Description	UCS (MPa)	UCS (MPa) of test specimen	Remarks
A	Very high strength	>224	9.62	plaster of Paris ,test specimen is classified as a very low strength Rock
B	High strength	112-224		
C	Medium strength	56-112		
D	Lower strength	28-56		
E	Very low strength	<28		

Table 6.16 Classification of intact Rock Materials based unconfined compressive strength after Stapledon and ISRM 1971

Term for UCS	Symbol	Strength (MPa)	UCS (MPa) of test specimen	Remarks
Extremely weak	EW	25-1	9.62	plaster of Paris ,test specimen is classified as weak Rock
Very weak	VW	1-5		
Weak	W	5-25		
Medium Strong	MS	25-50		
Strong	S	50-100		
Very Strong	VS	100-250		
Extremely Strong	ES	>250		

- The original classification due to Deere and Miller (1966) was suggested only for intact rocks.
- The table is an extended version of Deere and Miller and will cover very low strength to very high strength rocks.

Table 6.17 A Strength classification of intact rock after Ramamurthy and Arora 1994

Strength classification of intact rock after Ramamurthy and Arora 1994			Based on UCS test for the plaster of Paris , intact specimen	
Class	**Description**	**UCS (MPa)**	**UCS (MPa) of test specimen**	**Remarks**
A	Very high strength	>250	9.62	plaster of Paris ,test specimen is classified as low strength Rock
B	High strength	100-250		
C	Moderate strength	50-100		
D	Medium strength	25-50		
E	Low strength	5-25		
F	Very low strength	<5		

Table 6.18 A Strength of intact rock classification after BIENIAWSKI, 1971

Qualitative Description	Compressive Strength (MPa)	UCS (MPa) of test specimen	Remarks
Exceptionally strong	>250	9.62	plaster of Paris , test specimen is classified as Very weak Rock
Very strong	100-250		
Strong	50-100		
Average	25-50		
Weak	10-25		
Very weak	2-10		
Extremely weak	1-2		

Table 6.19 Summary of strength classification for plaster of Paris intact specimen

Strength classification proposed by previous researchers				**Strength classification based on present experimental study**	
Reference details	**Description**	**class**	**UCS range (MPa)**	**UCS (MPa)**	**Remarks**
Stapledon and ISRM, 1971	Weak	W	5-25	9.62	Weak rock
Bieniawski, 1971	Very weak	VW	2-10	9.62	Very weak
Ramamurthy and Arora,1994	Low strength	E	5-25	9.62	Low strength
Deere and Miller, 1966	Very low strength	E	<28	9.62	Very low strength

6.7.2 Classification of intact Lime-POP mix Specimen

Table 6.20 A Strength classification of intact rock after Deere and Miller, 1966

Class	Description	UCS (MPa)	UCS (MPa) of test specimen	Remarks
A	Very high strength	>224	8.77	Lime-POP mix test specimen is classified as a very low strength Rock
B	High strength	112-224		
C	Medium strength	56-112		
D	Lower strength	28-56		
E	Very low strength	<28		

Table 6.21 Classification of intact Rock Materials based unconfined compressive strength after Stapledon and ISRM 1971

Term for UCS	Symbol	Strength (MPa)	UCS (MPa) of test specimen	Remarks
Extremely weak	EW	25-1	8.77	Lime-POP mix test specimen is classified as weak Rock
Very weak	VW	1-5		
Weak	W	5-25		
Medium Strong	MS	25-50		
Strong	S	50-100		
Very Strong	VS	100-250		
Extremely Strong	ES	>250		

- The original classification due to Deere and Miller (1966) was suggested only for intact rocks.
- The table is an extended version of Deere and Miller and will cover very low strength to very high strength rocks.

Table 6.22 A Strength classification of intact rock after Ramamurthy and Arora 1994

Strength classification of intact rock after Ramamurthy and Arora 1994			Based on UCS test for the Lime-POP intact specimen	
Class	**Description**	**UCS (MPa)**	**UCS (MPa) of test specimen**	**Remarks**
A	Very high strength	>250	8.77	Lime-POP mix test specimen is classified as a low strength Rock
B	High strength	100-250		
C	Moderate strength	50-100		
D	Medium strength	25-50		
E	Low strength	5-25		
F	Very low strength	<5		

Table 6.23 A Strength classification of intact Rock Materials after BIENIAWSKI, 1971

Qualitative Description	Compressive Strength (MPa)	UCS (MPa) of test specimen	Remarks
Exceptionally strong	>250	8.77	Lime-POP mix test specimen is classified as a Very weak Rock
Very strong	100-250		
Strong	50-100		
Average	25-50		
Weak	10-25		
Very weak	2-10		
Extremely weak	1-2		

Table 6.24 Summary of strength classification of intact Lime-POP mix Specimen

Strength classification proposed by previous researchers				Strength classification based on present experimental study	
Reference details	**Description**	**class**	**UCS range (MPa)**	**UCS (MPa)**	**Remarks**
Stapledon and ISRM, 1971	Weak	W	5-25	8.77	Weak rock
Bieniawski, 1971	Very weak	VW	2-10	8.77	Very weak
Ramamurthy and Arora,1994	Low strength	E	5-25	8.77	Low strength
Deere and Miller, 1966	Very low strength	E	<28	8.77	Very low strength

Table 6.25 Classification of rock based on failure strain (T. Ramamurthy)

Strength classification of rock based on failure strain					Strength classification based on present experimental study	
Specimen details	**Description**	**Class**	**Failure Strain (%)**	**Failure Strain of specimen**	**UCS (MPa)**	**Remarks**
Plaster of Paris	Very high failure strain	a	> 2	3.947	9.62	Very high failure strain
Lime-plaster of Paris mix	Very high failure strain	a	> 2	3.947	8.77	Very high failure strain

CHAPTER - 7

CONCLUSIONS

From this study following conclusion can be listed:

1. The cohesion (c_j) and friction angle (Φ_j) for plaster of Paris specimen was found to be 0.178 MPa and 39^0 respectively.
2. On addition of lime to the plaster of Paris, the Cohesion (c_j) and friction angle (Φ_j) of Lime-POP mix specimen found as 0.169 MPa and 38º for respectively from direct shear test.
3. The Uniaxial compressive strength of plaster of Paris and Lime-POP mix intact specimens was found to be 9.62 MPa and 8.77 MPa respectively.
4. Empirical relationship of Arora (1987) and Padhy (2005) have been used to predicting the result of strength and elastic modulus of jointed rocks it seems that which is almost closer with the present experimental result of strength and elastic modulus values [Empirical relationship of Arora (1987), $\sigma cr = e^{-0.008*Jf}$ and $Er = e^{-1.15*10^{-2}*J_f}$] [Empirical relationship of Padhy (2005), $\sigma cr = e^{-0.09*Jf}$ and $Er = e^{-1.25*10^{-2}*J_f}$]
5. The strength of jointed specimen depends on the joint orientation β° with respect to the direction of major principal stress, and uniaxial compressive strength has found maximum at 90° and minimum at 30° (Ref. Table no- 6.4, 6.5, 6.11, 6.12 and fig no 6.9 , 6.16).
6. The values of compressive strength ratio ($\sigma_{cr} = \sigma_{cj}/\sigma_{ci}$) also depends on the joint orientation β°. This ratio is least at 30° and maximum at 90°.
7. As the number of joints increases, the uniaxial compressive strength decreases.
8. Strength, elastic modulus and strength ratio decreases while plaster of Paris mixes with Lime.

9. Based on the result of uniaxial compressive strength test plaster of Paris and Lime-POP mix intact specimen both are classified as a low strength rock by various researchers.
10. The Deere and Miller engineering classification, originally developed for intact rocks, has been found, after suitable modifications, useful in classifying jointed rocks as well.
11. Various classifications as RMR, Q, RMI and GSI systems requires field data so for a specimens prepared from artificial materials their classifications may be done on the basis of the failure strain.

SCOPE OF FUTURE STUDY

This project is an approach towards a better understanding for classification system for rock mass. This study can be accomplished with few additional features in future. Some of the future scopes are as:

- By using to numerical models.
- More and more studies can be done for multiple joints at various angles of orientation.
- Different software's can be used to analyse the experimental results for the rock mass classification systems.

CHAPTER - 8

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